THE

ONTARIO WATER RESOURCES

COMMISSION

INTERMEDIATE COURSE

FOR

SEWAGE WORKS OPERATORS

1968

INTERMEDIATE COURSE

FOR

SEWAGE WORKS OPERATORS

December 2nd to 6th, 1968

ONTARIO WATER RESOURCES COMMISSION

Sewage Operators' Courses

Course I

Basic	1961	
Basic - repeat	1961	October 16-20th
Intermediate	1962	March 5-9th
Intermediate - repeat	1962	December 10-14th
Senior	1963	April 22-26th
Senior - repeat	1963	September 16-20th

Course II

Basic	1963	November 18-23rd
Intermediate	1964	May 25-29th
Senior	1964	December 7-11th

Course III

Basic	1965	May 31 - June 4th
Intermediate	1965	December 6-10th
Senior	1966	April 25-29th

Course IV

Basic	1966	December 5-9th
Intermediate	1967	May 15-19th
Senior	1967	December 4-8th

Course V

Basic	1968	June 17-21st
Intermediate	1968	December 2-6th
Senior	1969	June 9-13th

Course VI

Basic	1969	December 8-12th
Intermediate	1970	June 8-12th

GLOSSARY OF TERMS

ABS	alkyl benzene sulphonate
asu	areal standard units
BOD5	5-day biochemical oxygen demand
C	centrigrade degree
cc	cubic centimeter
cfm	cubic feet per minute
cfs	cubic foot per second
COD	chemical oxygen demand
cm	centimeter
cu	cubic
deg (o)	degree
DO	dissolved oxygen
F	Fahrenheit degree
fpm	foot per minute
fps	foot per second
ft	feet
gal	gallon
gpcd	gallon per capita per day
gpd	gallon per day
gph	gallon per hour
gpm	gallon per minute
hp	horsepower
hr	hour
Igpm	Imperial gallons per minute
In	inch
l	litre
LAS	linear alkyl sulphonate
lb	pound
m	meter
mg	million gallon also milligram
mgd	million gallons per day
mi	mile
min	minute
mg/l	milligram per litre
MLSS	mixed liquor suspended solids
ml	millilitre
mm	millimeter
MPN	most probable number

GLOSSARY OF TERMS (Cont'd.)

ppb	parts per billion
ppm	parts per million
psi	pounds per square inch
rpm	revolution per minute
sec	second
sq	square
SS	suspended solids
SVI	sludge volume index
U.S. gpm	United States gallons per minute
VSS	volatile suspended solids
wt	weight
yd	yard

ONTARIO WATER RESOURCES COMMISSION

INTERMEDIATE SEWAGE WORKS OPERATORS' COURSE

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SEWAGE WORKS OPERATORS' COURSE
ONTARIO WATER RESOURCES COMMISSION

INTRODUCTORY STATEMENT BY MR. D. S. CAVERLY
GENERAL MANAGER, OWRC.

When the courses for sewage works operators were first initiated by the Ontario Water Resources Commission, it was recognized that these should fall into three categories - a basic course, an intermediate course and a senior course leading to a certificate of qualification.

Having attended the basic course, you are now embarking on the second course of this series designed to advance the operator further along the road to effective sewage plant operation. In this intermediate course different subjects from those dealt with in the primary course will be handled. The course will also be more advanced in content. The operator will be asked to listen closely to the lectures given and to study the material carefully as it will form the background for future training in his field of operation.

As in the case of the basic course, the operator taking this intermediate course will again be asked to take an examination at its conclusion. Following the intermediate course, arrangements will be made, after a period, for him to take the final course leading to a certificate of qualification. He will have merited this certification through experience and training, through attendance at these operator's courses and through the examinations which he will have written. It is essential that sewage plant operators be well qualified.

The importance of proper plant operation needs to be emphasized continually. This goal cannot be achieved by merely constructing a plant, then paying little attention to its operation. If the sewage treatment works, on which considerable sums of money have been spent by the municipalities of this Province, are to be effective

in controlling stream pollution, they must be operated at maximum efficiency. This is the challenge which faces you.

One of the basic objectives of the Ontario Water Resources Commission is the protection of our watercourses from the harmful effects of pollution. These courses are designed toward that end and for this reason both Commission staff and the facilities of this laboratory building are being made available for this training programme. Together we are working towards a common goal. We wish you success in the interesting and worthwhile work which you have chosen.

THE SIGNIFICANCE OF SEWAGE PLANT DISCHARGES ON STREAMS

D. M. Veal
Division of Laboratories

INTRODUCTION

In order to understand the effect of sewage plant discharges on a receiving body of water, one must first of all realize that streams are characterized by numerous interacting physical, chemical and biological features. Through these interactions, complex organic chemicals tend to stabilize by breaking down into simple compounds, and living organisms tend to establish food-chain relationships.

Let us illustrate these interactions by using a typical example. Suppose a piece of raw sewage is released to a stream; bacteria will immediately attack the sewage and break it down into simple substances, for example nitrate (NO_3) and carbon dioxide (CO_2). These decomposition products will be used by algae as food. Algae will then be eaten by microcrustaceans, microcrustaceans will be eaten by minnows, and the minnows will be eaten by trout. When any organism in the food chain dies, bacterial action decomposes the organism and the nutrients are re-cycled once again.

Clean-water streams are characterized by a wide diversity of aquatic life, with no one species predominating. Each living organism is kept under control by limitations on space, food supply, and through predation by larger forms. Thus. a natural balance is maintained.

A source of pollution, such as a sewage treatment plant effluent, can disrupt not only the chemical and physical structure of a stream, but it can upset the whole biological balance.

DISSOLVED OXYGEN

The most important condition in a stream is its oxygen balance. Oxygen is essential for practically all forms of life in a stream, and it is used in the purification and stabilization of waste materials. Normally, the stream is saturated with dissolved oxygen; it contains as much oxygen as the water can hold in solution. This balance is controlled at the surface of the water. If excess oxygen is present, some is released as bubbles. If insufficient oxygen is present, more dissolves from the air to bring the dissolved oxygen level up to saturation.

There is normally only a slight tendency for stream waters to use up oxygen, due to their low organic content. Most stream waters use up to 2 ppm or less dissolved oxygen during 5 days, some less than 1 ppm. This is easily supplied from the surface, and the stream is maintained in balance, saturated with dissolved oxygen.

Natural organic substances in sewage are nutrients, of the same types the humans require (carbohydrates, fats and proteins), the only difference being that in sewage they are already partly decomposed or digested. These organic nutrients are used as food by bacteria; the bacteria require oxygen to metabolize this food, and therefore oxygen is used up in proportion to the amount of organic material. The Biochemical Oxygen Demand (BOD) test measures this tendency of the organic material to rob the receiving stream of dissolved oxygen. Thus the higher the BOD, the greater the rate of oxygen uptake.

The effect of the discharge of a sewage effluent on the oxygen balance in a stream can be illustrated by a diagram in which the dissolved oxygen content of the receiving stream, measured at a series of downstream sampling points, is plotted in reference either to mileage downstream, or equally well, to days of flow. The characteristic form of the resulting graph gives it the name 'Oxygen Sag Curve'.

There are three possible forms of the curve. In the first, a small increase in BOD can be met by an equal increase in the rate of aeration in the stream, which

replaces the oxygen used up. The result is no change in the dissolved oxygen content, it remains saturated, and there is little damage to the stream. In this case, aerobic bacteria act in attacking the organic waste directly using oxygen and forming cell-body-building materials, gases (carbon dioxide (CO_2) etc.), waste products and energy (for example heat). This is the ideal condition to be aimed at in treating the sewage before discharge. No more organic matter or BOD is discharged than can be taken care of by the stream's aeration capacity alone.

The discharge of BOD above this capacity results in the second form of the curve. Here the organic materials start using up Dissolved Oxygen faster than aeration can replace it. The Dissolved Oxygen level falls. As the BOD is met, the remaining demand grows less. Finally, a point is reached where aeration can meet the demand, then more than meet it. Thus the Dissolved Oxygen level reaches a minimum then recovers to its original level of saturation (see figure 2).

The third and most severe condition results from discharging BOD above both the stream's capacity for aeration, and its reserves of dissolved oxygen. In this case, the Dissolved Oxygen level is reduced to zero. The entire stream becomes septic. Anaerobic bacterial action takes place and continues until the organic material is reduced to a point where the aeration rate, which has continued at its maximum, finally is able to cope with the remaining BOD. The Dissolved Oxygen content then recovers gradually to a saturated level.

Aeration Rate

The maximum aeration rate is not constant throughout a stream. The surface/volume ratio varies. Thus a narrow, deep section of the stream has less surface from which to obtain oxygen than a shallow, wide stream section. Diffusion is a negligible factor in dispersing the oxygen picked up at the surface throughout the depth of a stream. The most important factor of all is turbulence, as it is in your aeration tanks. A rapidly flowing turbulent section of a stream has many times the aeration

capacity of a quiet, sluggish section. Thus in any stream the recovery graph will vary with topography. Rapidly flowing shallow streams that my show no distinct depletions or even septicity, may exhibit oxygen depletion in calm, deep pools or impoundments some miles downstream from the source of pollution.

The stream may also be aerobic at the surface and still show oxygen depletions or septicity at or near the bottom, especially where there are sludge deposits. (see figure 5).

Seasonal Variations

The most critical conditions below a discharge of organic pollution occur during the summer. A number of factors are responsible, all of which reinforce one another. First, stream flows are at a minimum; there is less dilution for a given quality and volume of sewage discharge. The resulting BOD concentration in the stream will be higher. Temperatures are higher. This has a threefold effect. The Biochemical Oxygen Demand is exerted more rapidly due to the increased rate of bacterial metabolism. The saturation level for Dissolved Oxygen is lower, since as the temperature rises, less oxygen is held in solution. At higher temperatures, fish become more susceptible to damage from low oxygen levels. With lower flows, streams are sluggish and aeration capacity from turbulence may decrease.

The total end result is that any adverse condition in the stream is intensified profoundly during the summer.

THE FOUR ZONES BELOW A SOURCE OF ORGANIC POLLUTION

The most damaging materials in sewage are the putrescent organic substances and the suspended solids. The impact of organic materials in exerting a BOD demand on the stream has already been described. The effect of suspended sewage solids is threefold. They impart objectionable turbidity to the water. They settle out in slow flowing sections of the stream to form sludge beds. Unless treatment

is complete, they contain a large proportion of organic material which causes putrescence or anaerobic digestion in these sludge beds.

Some streams are damaged simply by soil particles being washed from farmers' fields. These cause turbidity and settle out as silt beds, but due to their lower organic content do not putrefy as readily as do sewage sludge beds.

For purposes of illustration, a hypothetical stream seriously impaired by sewage wastes is often divided into four zones:

1 Zone of degradation.
2 Zone of active decomposition.
3 Zone of recovery.
4. Zone of clear water.

<u>1) Zone of degradation</u>

The zone of degradation is characterized by the presence of large quantities of undecomposed or slightly altered constituents of sewage, ample oxygen in the upper portion facilitating aerobic bacterial life, and turbid water hindering growth of algae. Fecal bacteria are numerous, indicated by coliform populations over 10,000 per ml., with an initially low number of microscopic animals (protozoans).

Rapid decomposition sets in and continues until oxygen and food decrease sufficiently to become limiting. Coincident with decomposition, carbon dioxide content increases and larger plants and animals disappear. For example, stonefly and mayfly nymphs, which live on the stream bottom, are wiped out because of their sensitivity to pollution. Desirable game fish such as trout, swim away from this part of the stream. On the other hand, tolerant organisms such as sludgeworms and bloodworms now have less competition and more food material, and they increase in number.

Microscopic animals, notably ciliated protozoans, increase and may reach populations of several thousands per milliliter (see graph number 6). Here they feed on the growing bacterial population until they fall victim to rotifers and crustaceans, the larger microscopic animals.

The efficiency of the sewage consuming biological machine depends on a close-knit savage society in which one organism captures and eats another. Bacteria do a more rapid job of breaking down sewage when several species are present and actively multiplying. However, stabilization is increased markedly when voracious bacteria-eating ciliates are introduced, keeping the bacterial population actively multiplying. The role of protozoans in the decomposition of sewage is not completely defined, but it is known that they may also feed on sewage particles as well as on bacteria.

At the same time that this activity is progressing in the aerobic layer, an equally active anaerobic microbial population is attacking the oxygen-depleted settled sludge, over which a "sewage-fungi" developes - this name is applied to any visible, filamentous, thread-like growth observed in sewage polluted water that does not possess a green color. (A greyish cotton-like mass of threads attached to rocks, limbs or to any debris). It varies greatly in composition depending on environment and may be composed of certain of the filamentous bacteria, filamentous algae and true fungi (see graph number 5).

Of the bacteria, Beggiatoa and at times Leptothrix and Crenothrix are encountered. The true fungi are represented by filaments of Leptomitus, Achyla and Saprolegnia, the latter often found growing on dead animals rather than plant tissue.

Oscillatoria, a blue-green filamentous algae and Chara and Spirogyra, grass green filamentous algae, are occasionally found in material called "sewage fungi". Their green color may be masked by encrusted slimes and they are frequently mistaken for fungi.

These "fungi" rely almost entirely on organic matter for nutrients and are most efficient in decomposing some of the more resistant materials such as cellulose and waxes. As an example of the simple needs of fungi, mildew (caused by a group of fungi) can grow readily on damp leather shoes where nutrients are at a minimum and where even bacteria cannot exist. These "fungi" are generally aerobic and will not be active when the oxygen level is low (see graph number 5).

At the sewage discharge they will increase rapidly forming the first large population of organisms to attack the sewage in the stream. Most of the organic matter they break down is utilized to build cell "tissue", releasing by-products in a form easily decomposable by the next dominant population - the bacteria. The mass of filaments left behind after conditions become unfavourable for fungal growth due mainly to a lack of oxygen, are also easily digested by bacteria (see graph number 4).

The zone of degradation persists until the oxygen falls to a level of about 3.5 ppm. Then the zone of active decomposition commences.

2) Zone of active decomposition

This zone continues until the oxygen (O_2) drops to a minimum and again returns to 40% of saturation (3.5 ppm O_2). This area is often referred to as a "septic area".

Because of the lack of oxygen, aerobic bacterial activity is replaced by an active population of anaerobes. The end products of aerobic and anaerobic decomposition of organic matter are almost entirely different as shown below.

Aerobic	Anaerobic
(using dissolved oxygen)	(avoiding using dissolved oxygen which is absent or scant)
(Carbon) C-->1-->CO_2 (Carbon dioxide)	C-->1-->CH_4 (methane) (some CO_2)
(Hydrogen) H-->1-->H_2O (water)	H-->1-->H_2 (gaseous hydrogen)
(Sulphur) S-->1-->SO_4 (sulphate)	S-->1-->H_2S (sulphides)
(Nitrogen) N-->1-->NO_3 (nitrate)	N-->1-->NH_2, N_2 (amines, gaseous nitrogen)
(Phosphorus) P-->1-->PO_4 (phosphate)	P-->1 ? not known
1 = many intervening steps.	1 = many intervening steps

Anaerobic decomposition leads to the production of foul smelling organics as ammonia (NH_3) and hydrogen sulfide (H_2S).

Toward the end of this zone, bacterial activity decreases markedly due to antagonistic elements (limited food and increase in number of protozoans), and is replaced by that of larger forms of life, such as algae (see graph number 6).

Needless to say, most of the larger animals are not found in this zone because of a lack of oxygen. Blackfly larvae, mayfly and stonefly nymphs, scuds and fish either suffocate under these conditions, or move out to a more suitable environment. Only the animals which can tolerate low oxygen tensions remain. Sludgeworms, for example, are specially adapted by having body fluid containing a substance like haemoglobin in our blood that has a great affinity for oxygen. These worms bury the front part of their bodies in the mud, and swing their tails back and forth in the water to circulate the water and therefore make use of every available bit of oxygen. Because of the lack of predation from other animals and with the excess food supply, these worms often become so numerous that they color patches of the stream bottom red. Similarly, "bloodworms" which are actually larvae of midges (mosquito-like insects), are quite tolerant and become abundant. Other animals cope with the oxygen problem by breathing air directly. The rattailed maggot has a long tube extending to the water's surface through which it breathes. Some snails can regulate their buoyancy and surface now and then for air. However, only a few of the specialized animals can survive, resulting in a small variety of only the most pollution-tolerant organisms.

3) Recovery zone

In the recovery zone, the dissolved oxygen concentration returns to near normal. Most of the organic materials with a high oxygen demand were decomposed in the previous zones, and in this zone surface aeration can almost keep up to oxygen consumption. Aerobic bacteria again take over the role of decomposition, although the total bacterial population decreases because of the decrease in food. The water is less toxic to algae in this area and rich in nutrients and heavy blooms of blue-green algae often develop in a slow moving stream.

The variety of animal life increases, and the sludgeworm populations decrease. The sow bug and the leech are depicted here and like the earlier forms they too come in considerable numbers. For instance, it is not uncommon to find a dozen leeches under every stone in the recovery area. As the stream recovers more and more species appear, dragonfly nymphs become common and later scuds appear. Hardy fish move up and down this section feeding when conditions are good and moving out when the oxygen drops. Usually there are coarse fish like carp and minnows and it is not until the stream is almost fully recovered that the desirable game species return.

4) Zone of clear water

The characteristics of this zone are similar to those above the sewage plant effluent. Most of the organic materials have been mineralized in the zones of degradation, active decomposition and recovery. The biota is made up of a wide variety of species and pollution-intolerant organisms are commonly found. Desirable game fish such as trout are again found because of the high dissolved oxygen content and availability of fish-food organisms (stoneflies, mayflies, etc.). An ecologist would refer to this zone as having a "well-balanced biota", meaning that many different organisms are sharing in the total biological productivity of the stream.

This description of the four zones refers to the worst condition. We hope that with an

efficient well-run sewage treatment plant, the receiving stream will never reach this deplorable condition. Quite often, little or no oxygen sag is detected, in which case the stream can act as a purification system with little damage resulting. The extent of damage to the stream naturally depends on the volume and characteristics of the effluent as well as on the volume and characteristics of the stream.

NON-DEGRADABLE SUBSTANCES

An example of synthetic organic materials, many of which are resistant to biological treatment, is synthetic detergents, or surfactants, which until recently contained a very stable chemical known as alkyl benzene sulfonate (ABS). You are probably all acquainted with the foaming effect these detergents have on the sewage plant and receiving water. Fortunately, the major detergent industries now substitute the alkyl benzene sulfonate for a linear alkyl sulfonate, which is bio-degradable. However, phosphorus is still released after degradation, thus providing a nutrient for algal growth.

Many sewage treatment plants also contain industrial effluents which contain non-degradable components. Chemicals such as heavy metals, pesticides, phosphates and nitrates are released to the stream unchanged by treatment. The pesticide DDT for example can readily accumulate to levels which are toxic to fish and other aquatic life.

THE NUTRIENT PROBLEM

Unfortunately, problems can be encountered in receiving waters even after sewage wastes have undergone primary and secondary treatment. Even after most of the solids and BOD has been removed, soluble forms of nutrient elements such as phosphorus and nitrogen are released to the stream, and these act as nutrients for algae. One must remember that the activated sludge process converts organic phosphorus compounds into a soluble form which is released as a soluble fertilizer. A resulting algae bloom may clog water intake filters further downstream, or it may produce a taste or odour in the water which will interfere with other water users. Similarly, rooted aquatic plants may flourish

as a result of these nutrient materials and interfere with boating and other recreational activities. If plant production becomes excessive, high BOD's will be created when the algae and rooted plants die off in the fall and decay. This increases the bacterial activity, and in fact, the production of the total mass of plants and animals will increase.

While such things as algae blooms and resulting tastes and odours are problematic, perhaps a more important consideration is the fact that "aging" of the receiving water is accelerated through man's addition of nutrients, thereby spoiling the receiving waters for other uses 'before their time'. It is clear that nutrient control through tertiary treatment should be the objective which hopefully will be realized in the near future. Chemical methods are already known whereby phosphorus can be settled out of solution, but they are expensive and further work is needed to formulate cheaper and more efficient techniques.

CONCLUSION

One of the most important aspects of a stream or lake is its dissolved oxygen. For this reason, effluents from sewage treatment plants should be as low as possible in BOD. It is essential to point out that the biota of a stream will not reflect the average condition of your effluent but the worst condition that has occurred in the past several months. Life in a stream must continue to breathe just as people, and while it can be said that in a year people breathe on an average of thirty times a minute this is not of much importance if for 10 minutes during the year there was no air to breathe.

With this thought in mind, therefore, every precaution should be taken to prevent by-passing, pumping of sludge, over-chlorination or any other operation that will cause a critical stream condition. This is of particular importance in the warm water season when available dilution is at a minimum and the quantity of oxygen in the water at a minimum. When shutdowns are necessary, try to

schedule them for spring or wet weather or even at night when the sewage strength is weak. Remember that your good work for several months previous can be wasted when one batch of raw sewage or sludge escapes proper treatment.

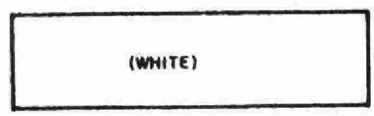
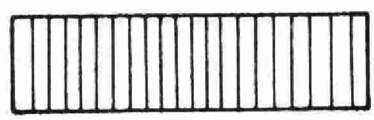
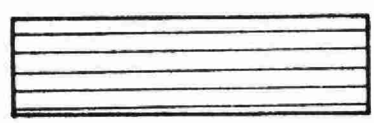
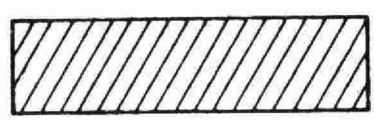
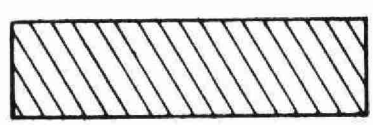
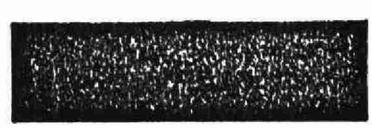
LEGEND FOR DIAGRAMS FIGURE 1-8 AS REPRODUCED FROM THE PUBLIC WORKS MAGAZINE FOR JULY 1959 FROM THE ARTICLE:

"STREAM LIFE AND THE POLLUTION ENVIRONMENT"

BY

A.F. BARTSCH, ASSISTANT CHIEF FOR SPECIAL TECHNICAL SERVICES

W.M. INGRAM, IN CHARGE, BIOLOGICAL FIELD INVESTIGATIONS FOR WATER POLLUTION CONTROL, WATER SUPPLY AND WATER POLLUTION RESEARCH, R.A.TAFT SANITARY ENGINEERING CENTER, U.S. PUBLIC HEALTH SERVICE, CINCINNATI, OHIO.

COLOURS IN ORIGINAL DIAGRAM	TRANSFERRED TO BLACK HATCHING IN REPRODUCED DIAGRAM
GREEN	
ORANGE & YELLOW	
RED	
BLUE	
GREY	
BLACK	

J.R

THE ENVIRONMENT

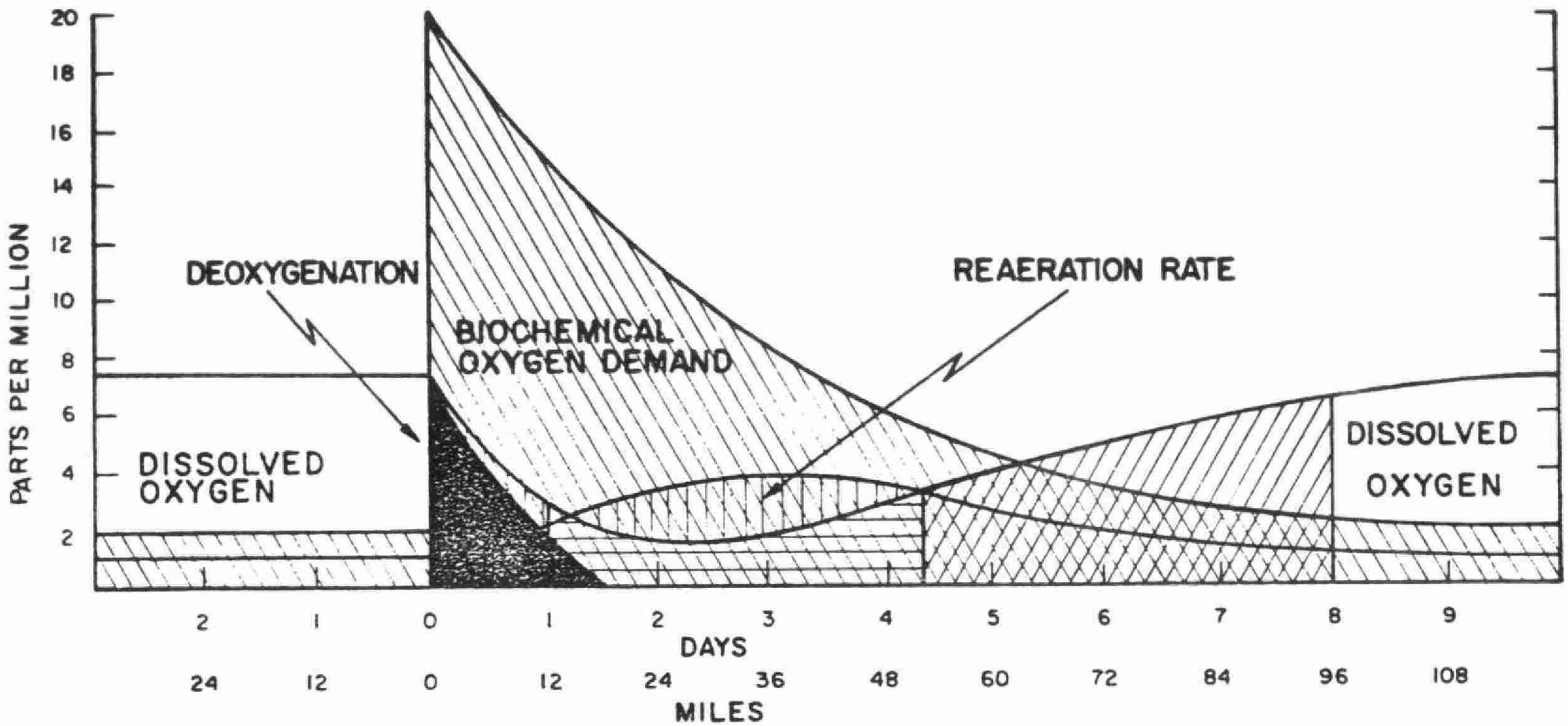


FIG. 2 THE DISSOLVED OXYGEN CONCENTRATION IN THE STREAM IS PARTIALLY DESTROYED BY THE POLLUTION LOAD. FULL DEPLETION IS AVOIDED BY REAERATION PROCESSES.

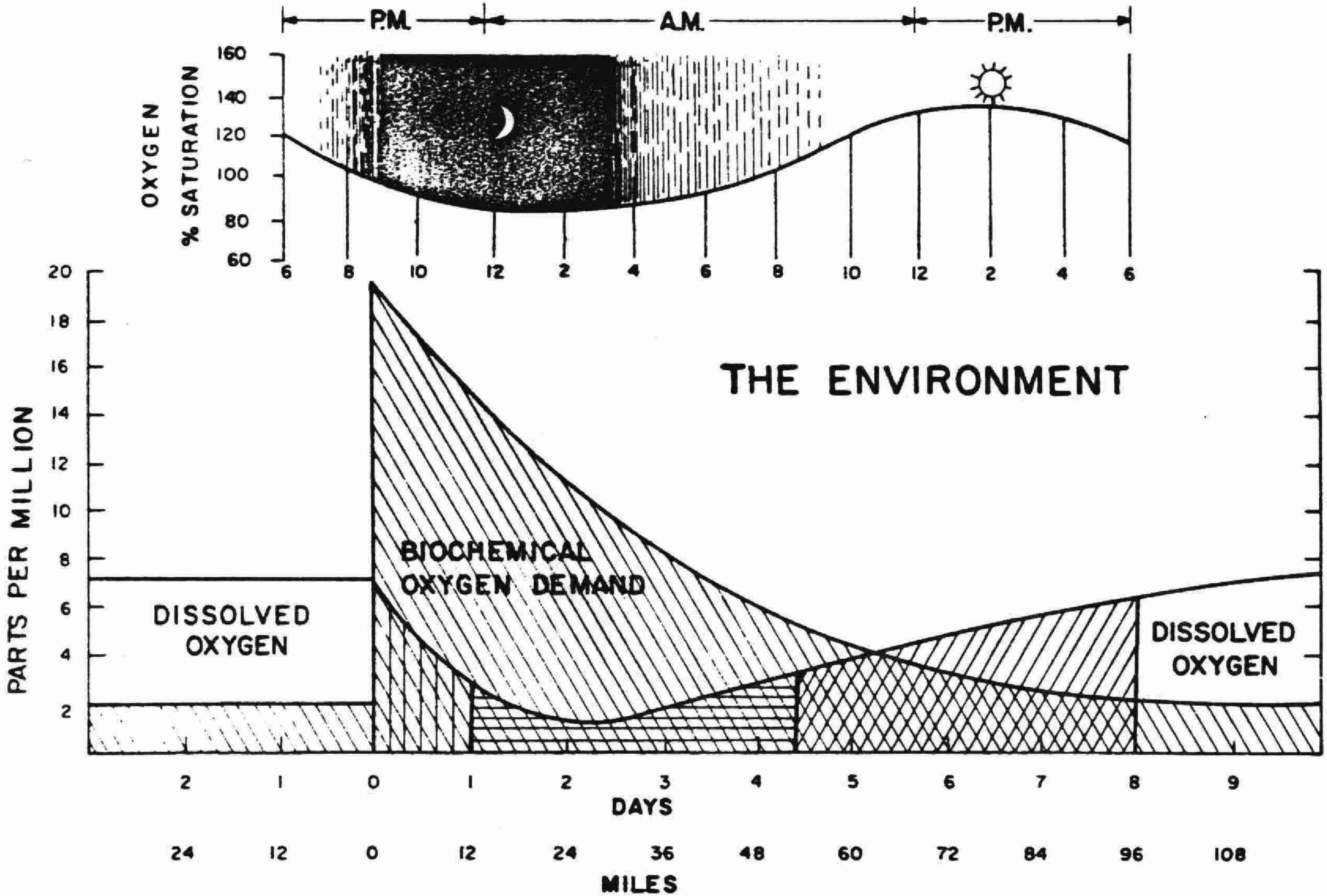


FIG. 3 DISSOLVED OXYGEN FLUCTUATES ACCORDING TO AVAILABLE LIGHT, A RESULT OF PHOTOSYNTHESIS. THUS, VALUES ON THE LOWER CURVE ARE SUBJECT TO DAILY VARIATION.

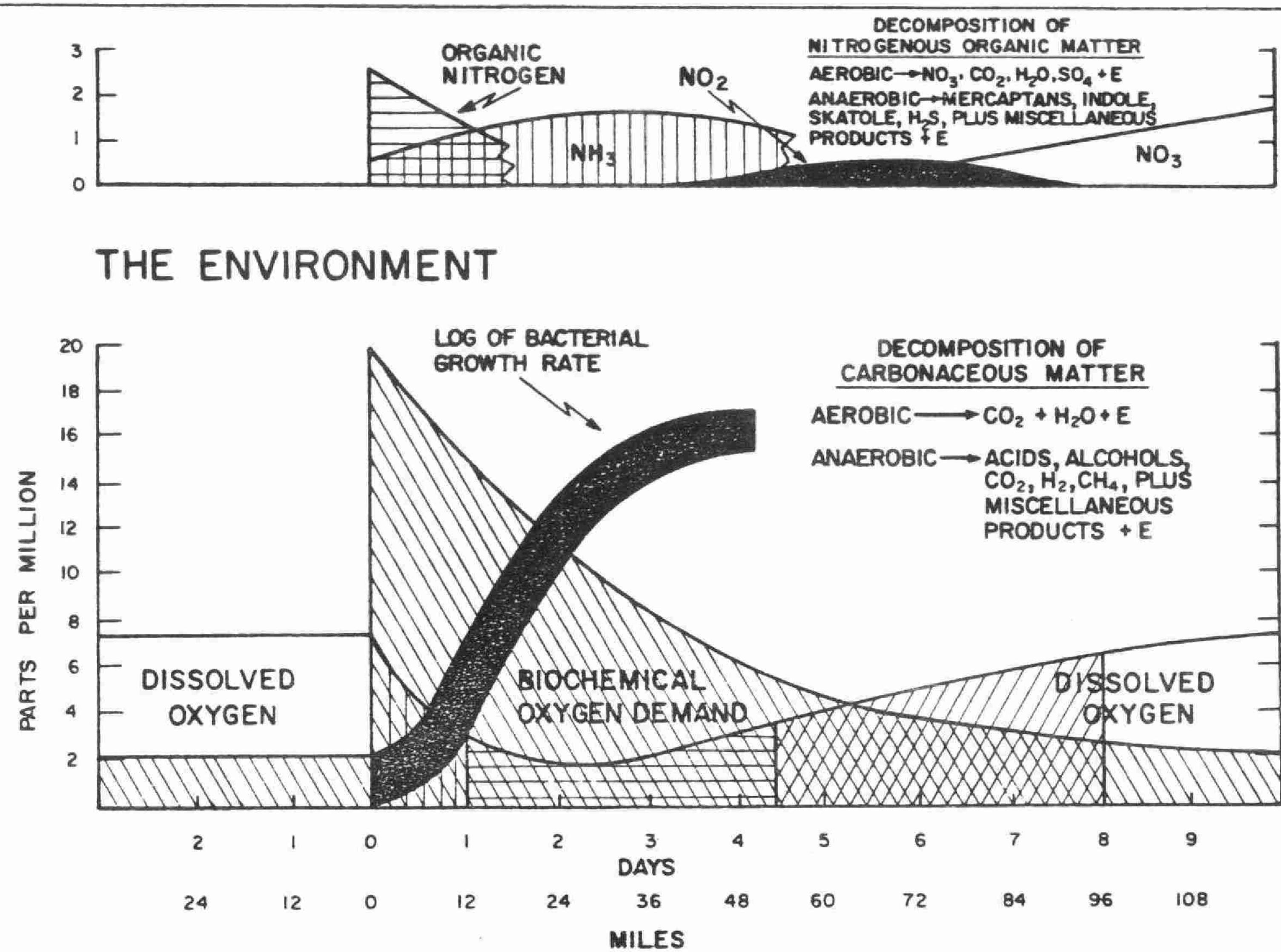


FIG. 4 WITH A HEAVY INFLUX OF NITROGEN AND CARBON COMPOUNDS FROM SEWAGE, THE BACTERIAL GROWTH RATE IS ACCELERATED AND DISSOLVED OXYGEN IS UTILIZED FOR OXIDATION OF THESE COMPOUNDS. AS THIS PROCEEDS, FOOD IS 'USED UP' AND THE B.O.D. DECLINES.

THE BIOTA

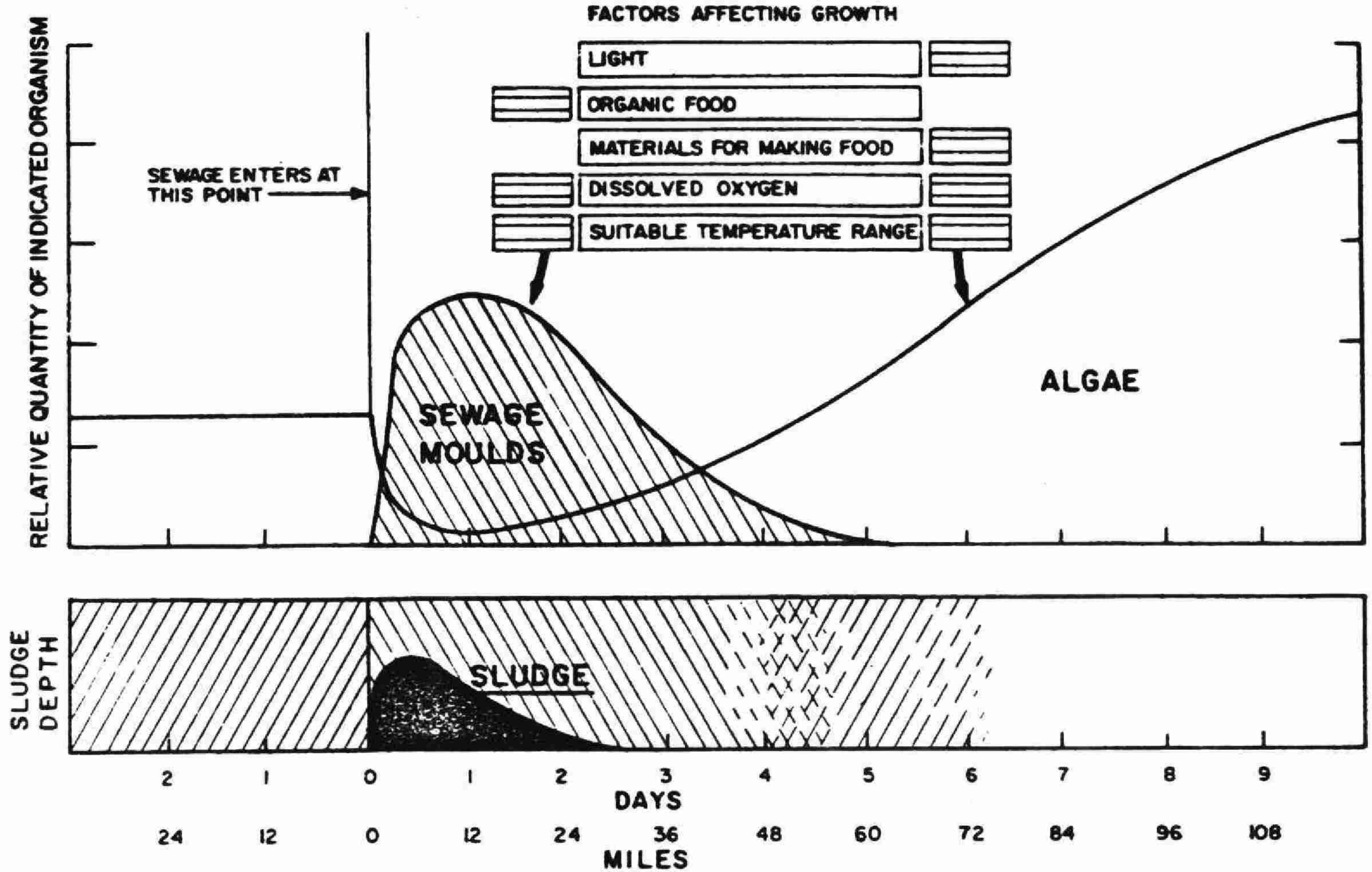


FIG. 5 SHORTLY AFTER SEWAGE DISCHARGE, THE MOULDS ATTAIN MAXIMUM GROWTH. THESE ARE ASSOCIATED WITH SLUDGE DEPOSITION SHOWN IN THE LOWER CURVE. THE SLUDGE IS DECOMPOSED GRADUALLY; AS CONDITIONS CLEAR UP, ALGAE GAIN A FOOTHOLD AND MULTIPLY.

JR

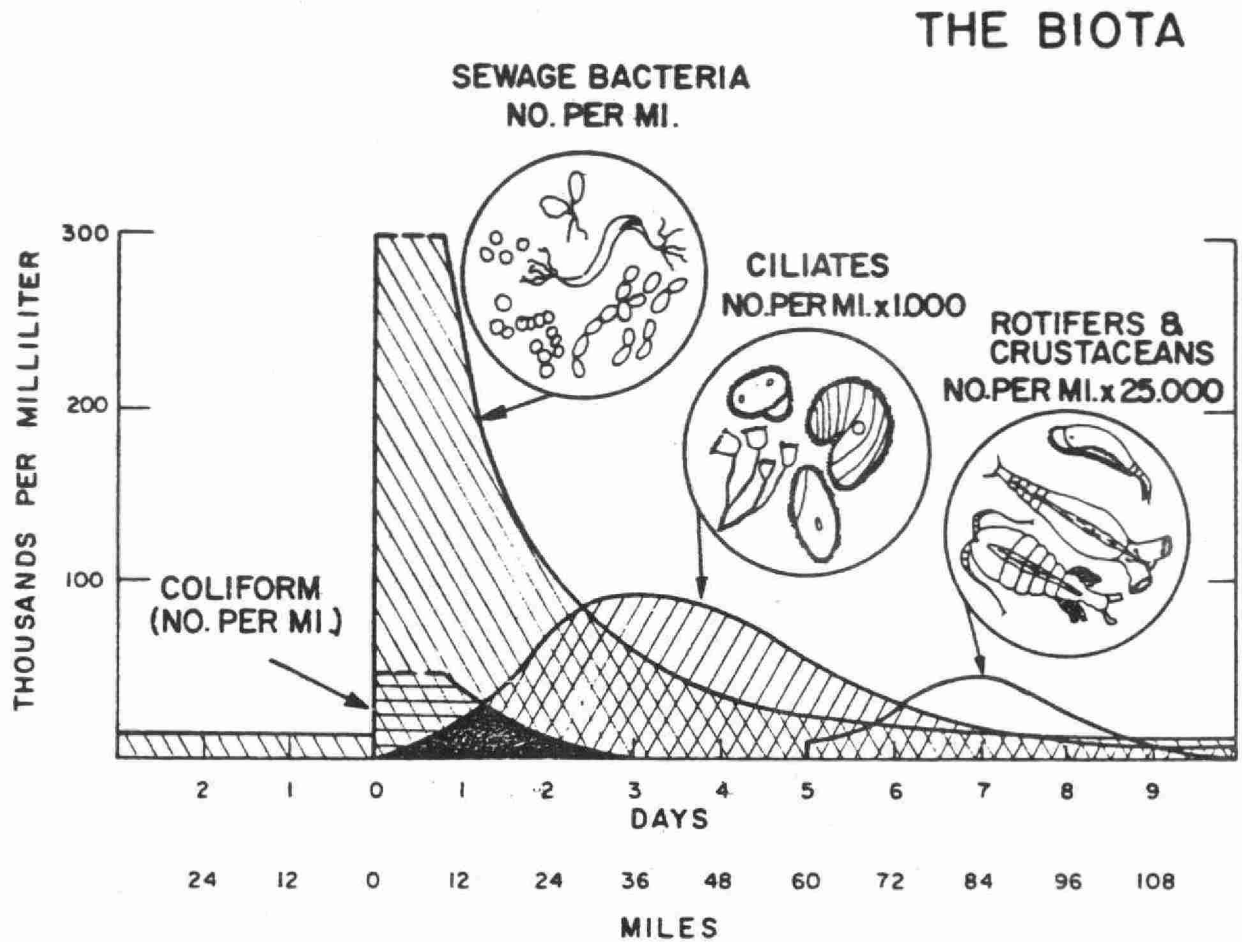


FIG. 6 BACTERIA THRIVE AND FINALLY BECOME PREY OF THE CILIATES, WHICH IN TURN ARE FOOD FOR THE ROTIFIERS AND CRUSTACEANS.

THE BIOTA

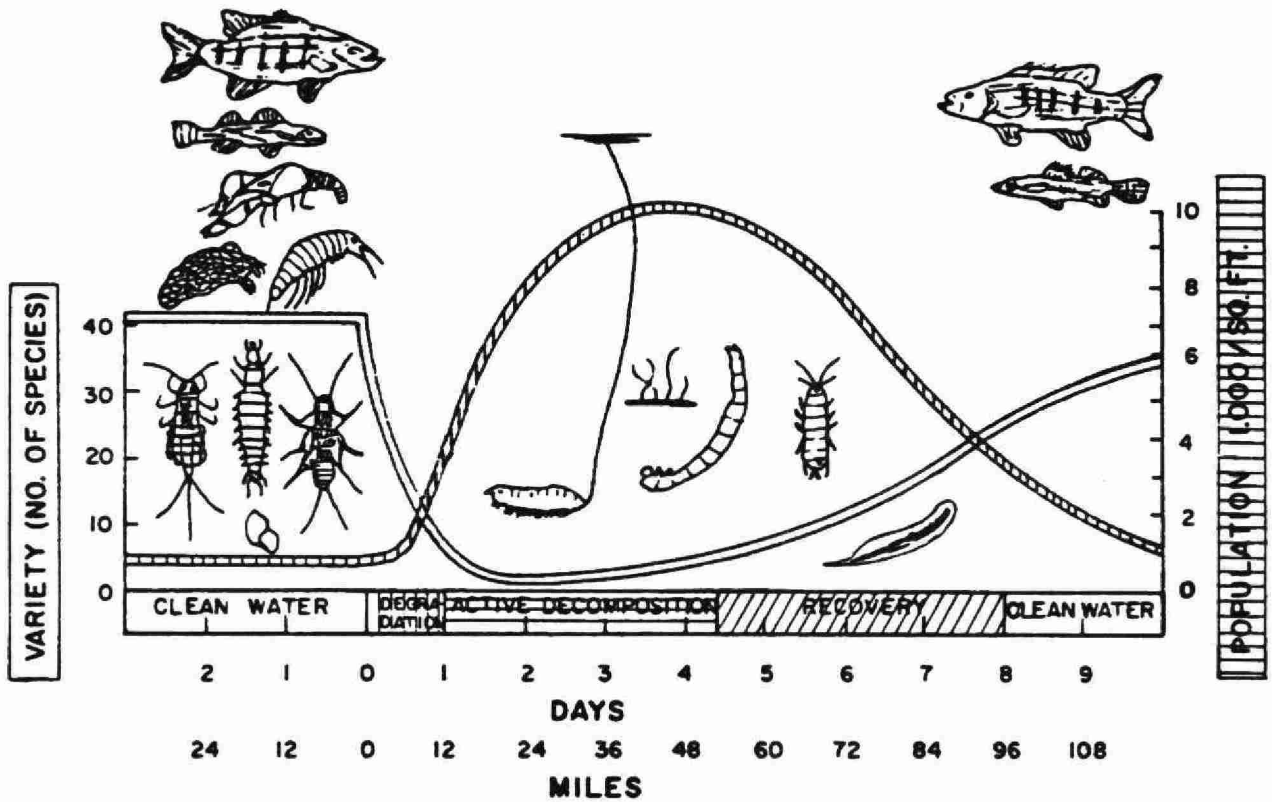


FIG. 7 THE UPPER CURVE SHOWS THE FLUCTUATIONS IN NUMBERS OF SPECIES; THE LOWER CURVE, THE VARIATIONS IN NUMBERS OF EACH.

J.R.

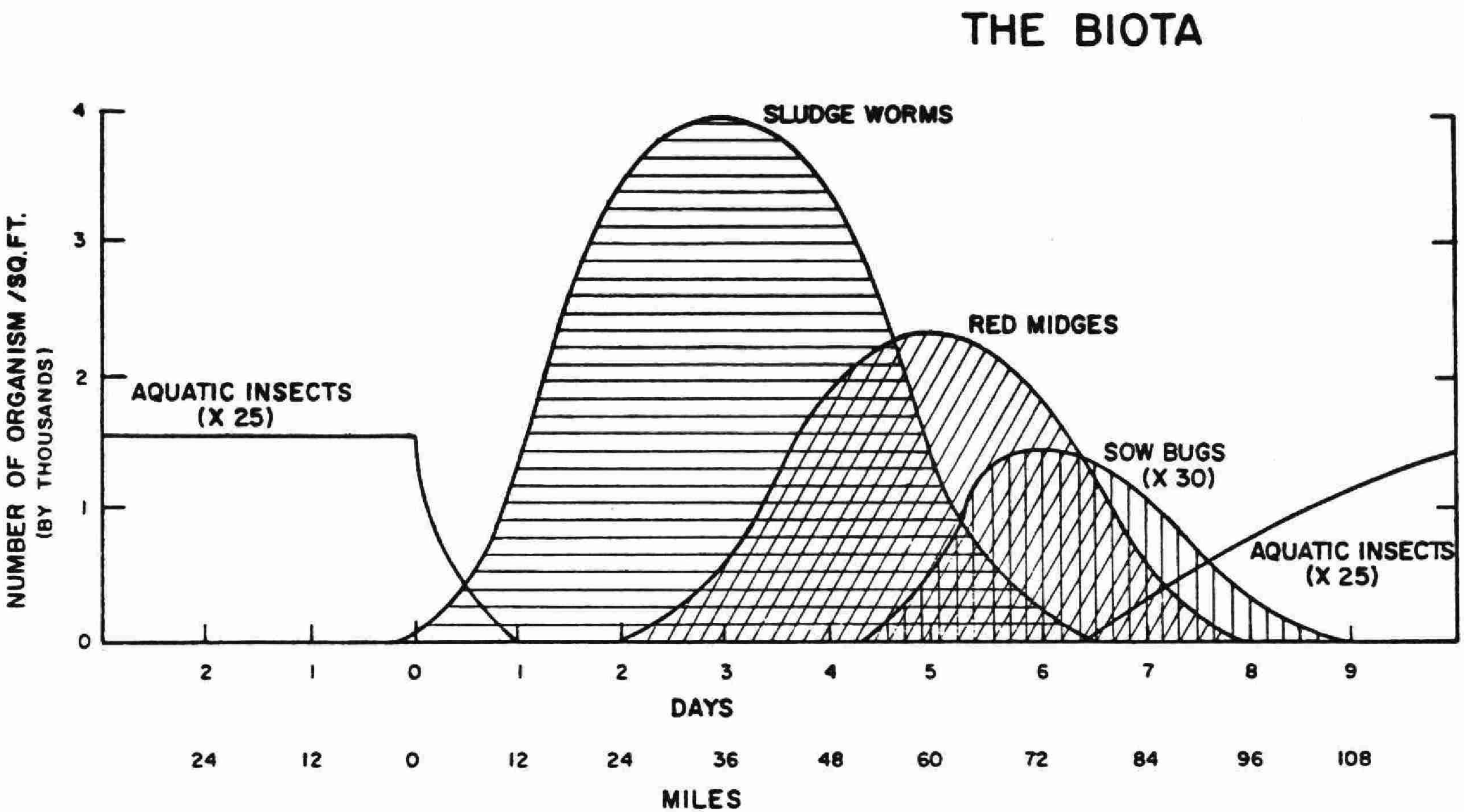


FIG. 8 THE POPULATION CURVE OF FIGURE 7 IS COMPOSED OF A SERIES OF MAXIMA FOR INDIVIDUAL SPECIES, EACH MULTIPLYING AND DYING OFF AS STREAM CONDITIONS VARY.

OPERATION AND MAINTENANCE OF SEWAGE PUMPING STATIONS

A. E. Symmonds
Maintenance Engineer

PURPOSE

Sewage pumping stations are an integral part of most pollution control facilities. Only in rare instances is all sewage collected and conveyed to treatment facilities by gravity thus the pumping station is for the most part an irreplaceable link in the chain.

It is the purpose of this lecture to describe to you the most common types of sewage pumping stations encountered, outlining peculiarities, operating difficulties, and specific maintenance requirements.

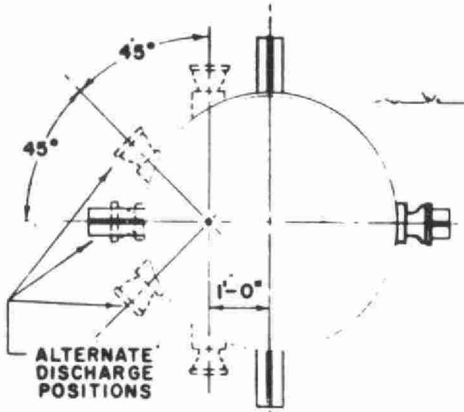
GENERAL

Basically, there are four common types of pumping stations in use today. These are as follows:

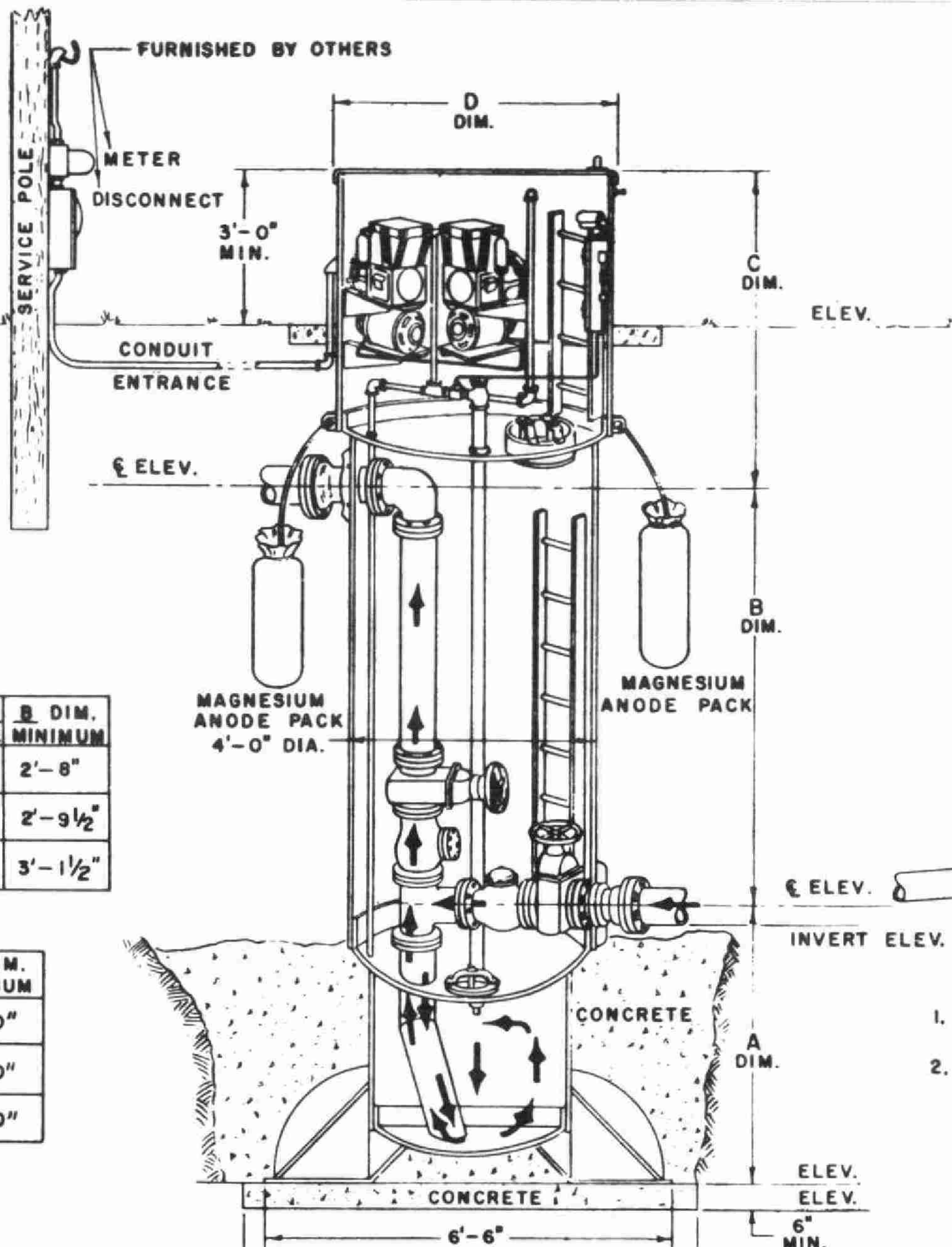
1. Ejector station
2. Stations utilizing submersible pumps
3. Factory built stations using conventional centrifugal pumps.
4. Field constructed or custom built pumping stations.

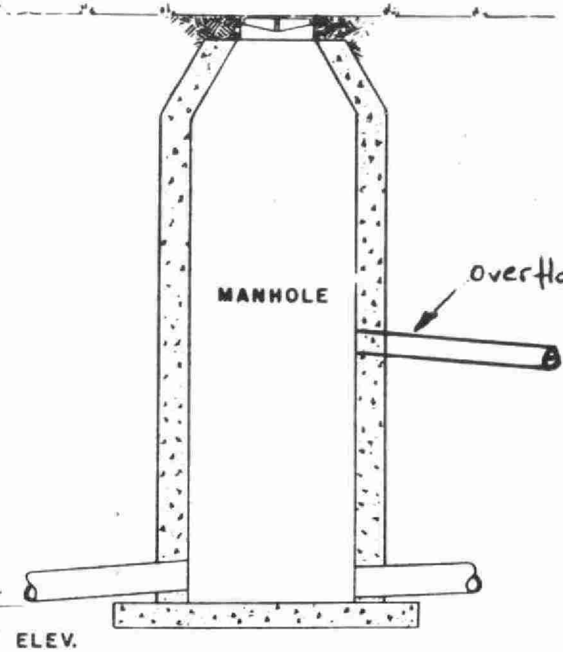
Naturally, there are many variations of the basic principle of each of these types, but I believe in most cases, you will find that a general knowledge will put you in a good position to understand these variations when they are encountered.

In the following material, I will deal individually and in depth with the first three types of stations. The



STATION PLAN





GPM	A DIM. MINIMUM
0-100	3'-10"
101-150	4'-9½"
151-200	5'-9"

PIPE SIZE INLET	PIPE SIZE DISCHARGE	B DIM. MINIMUM
4" DIA.	4" DIA.	2'-8"
6" DIA.	4" DIA.	2'-9½"
6" DIA.	6" DIA.	3'-1½"

COMPRESSOR	C DIM. MINIMUM	D DIM. MINIMUM
9LD-1 THRU 9LN-10	7'-0"	5'-0"
9LQ-10 THRU 9LR-15	8'-0"	6'-0"
9LS-15 THRU 9LS-20	8'-6"	6'-0"

NOTE:

WHEN THE COMPRESSOR SELECTION IS MADE ABOVE THE M-O-J MINIMUM LENGTH CURVE OF THE SELECTION CHART, THE MINIMUM LENGTH OF DIMENSION B OR DIMENSION C MUST BE INCREASED BY THE DIMENSION SHOWN ON THE NEXT HIGHER CURVE TO PROVIDE SUFFICIENT AIR STORAGE.

NOTE:
1. INTERIOR DETAILS OF CONSTRUCTION ARE TYPICAL BUT MAY VARY SLIGHTLY DUE TO SIZE OF STATION.
2. MATERIALS AND DIMENSIONS SHOWN THAT ARE NOT DIRECTLY APPLICABLE TO S&L EQUIPMENT ARE RECOMMENDATIONS ONLY, AND ARE NOT INTENDED TO CONFLICT WITH GENERAL SPECIFICATIONS.

GENERAL NOTES

1. USE CLASS 150 CAST IRON INFLUENT LINES.
2. DISCHARGE AND INFLUENT LINES SHOULD BE LAID ON CAREFULLY COMPACTED BACKFILL OR SUPPORTS TO PREVENT BREAKAGE BY SETTLEMENT

Pneumatic Ejector Station

custom built is very similar in mechanical design and operation to the third listed type and so basic principles of operation and maintenance will apply to both.

I. EJECTOR STATION

General

Where design flows are 100 GPM or less, pneumatic ejector stations have been used with considerable success.

In describing the operation of an ejector station one might very simply say that sewage flows into a sewage receiver by gravity and is discharged from the receiver and through the forcemain by means of compressed air.

During the filling cycle, the inlet check valve is held open by a positive sewage head, and the sewage receiver fills while the displaced air is vented to atmosphere. When the sewage reaches the electrode in the top of the receiver the three-way valve closes the vent line and opens the air discharge line allowing air to pass from the air reservoir to the sewage receiver. The compressed air forces the sewage out of the receiver and up the forcemain. The duration of the ejector cycle may be controlled in several ways, the two most common being:

1. A second electrode which is located at the bottom of the receiver. When the sewage level drops below this electrode, the ejection cycle is terminated.

2. A timed ejection period. Through testing it is determined just how long a period of time is necessary to completely empty the sewage receiver.

Having described the basic concept behind the operation of an ejector station, let us look at the ejector in a little more detail.

Influent works

Rather than a wet-well as found with conventional

pumping stations, it is more common to have an overflow manhole on the influent sewer to the ejector sewage receiver. During normal operation, the sewage flows directly through the manhole into the receiver. However, on failure of the ejector, sewage backs up in the manhole and eventually flows out a gravity overflow line which should be located at a low enough elevation to avoid flooding of sewer connections made to the influent sewer. It is a wise idea to incorporate where possible, a basket screen in this manhole. This will catch foreign objects which could pass down the influent line and jam the influent check valve.

Check Valve (influent)

The influent check valve is an essential piece of equipment. If this is jammed or inoperative, the station is useless. Within the past year or so, it has come to our attention that operating stresses on the clapper assembly within the valve are quite severe and in fact, have resulted in several failures. Since the station is useless until replacement parts are installed, it would be very wise to keep these parts on hand at all times.

A broken or severely damaged influent check valve is easily recognized by the discharge of sewage and air into the influent manhole during an ejection cycle.

Air Supply

Air is supplied by at least one compressor located within the ejector station. If possible, two compressors should be available, both being capable of providing sufficient air to operate the station at capacity. Air from the compressors is stored in an air storage tank, the pressure being maintained in this tank by high and low level pressure switches which start and stop the compressor at pre-selected pressures. Very little problem is experienced with this part of the control system. The compressors are designed for a relatively long operational life and as long as they are not being overworked and as a result overheated, maintenance and repair will be very low.

Maintenance of the compressor is usually limited to a close check on the oil level in the crankcase. It should always be kept at the prescribed level. Quincy compressors recommend that the oil be changed every 300 hours and the correct grade and quality of oil be used. If in doubt, check with the manufacturer. It is probably a good idea to plan on a routine replacement of the V-belts connecting the motor and compressor. Never replace just one belt.

Control Circuit

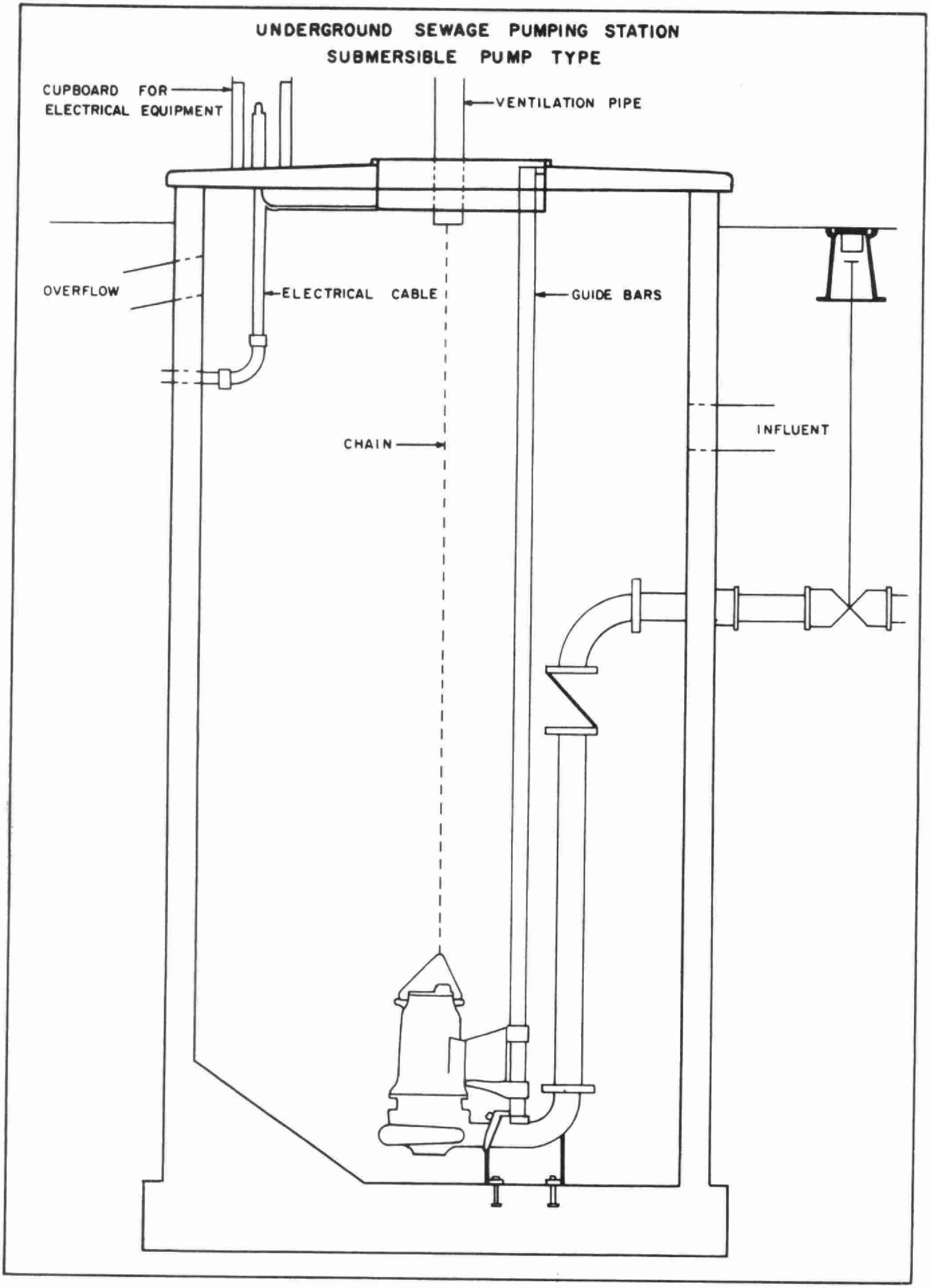
Ejectors are normally controlled by electrodes in the sewage receiver. As previously mentioned, the controls may consist of either a single electrode and a timer or two electrodes, one for starting the ejection cycle and one for stopping the ejection cycle. Most control circuits utilize direct current. A D.C. relay is energized by the completion of a circuit from the top of the probe to ground utilizing the sewage as the conductor. This relay operates either a three-way valve or two two-way valves which allow air to flow from the air storage pot to the sewage receiver. In the case of Smith & Loveless stations, the ejection cycle is based on an electrical timer setting. This timer will keep the three way valve energized for a predetermined period of time. When this time is elapsed, the valve is de-energized and returns to its "receiver vented" position.

The two basic failures which normally occur with the electrode control system are:

1. Electrode becomes coated with grease and mineral deposits which act as insulating material and prevent completion of the D.C. circuit.

2. The probe is grounded (in the case of Smith & Loveless stations the light on the control panel is out).

In both of the above cases, the best way to eliminate the problem is to clean the electrode tip.


UNDERGROUND SEWAGE PUMPING STATION
SUBMERSIBLE PUMP TYPE
CUPBOARD FOR ELECTRICAL EQUIPMENT
VENTILATION PIPE
OVERFLOW
ELECTRICAL CABLE
GUIDE BARS
INFLUENT
CHAIN

Forcemain and Discharge Check Valve

The operating stresses on the discharge check valve do not seem to be as great as those on the influent valve. Thus, it is not common to have problems with this piece of equipment. As far as the forcemain is concerned, it is possible to lodge some debris in this pipe (almost always 4 inches in dia.) and create a total dynamic head greater than the design head of the station. In a case such as this, the station will continue to try to eject as long as sewage is shorting the probe. In cases where a plugged or partially plugged forcemain is suspected, it might be possible to clear this by building up higher than normal pressures in the air storage chamber by altering the air pressure control switch.

II. PUMPING STATIONS UTILIZING SUBSERSIBLE PUMPS

General

The significant feature of this type of pumping station, is the utilization of submersible pumps and thus, in most cases, the elimination of a dry well. I say in most cases because it is most common to find the pumps mounted in a wet well with the controls for these pumps mounted on a pole above ground. However, where the installation is to be installed below a travelled roadway for example, and the above ground controls must be eliminated, it is possible to buy a package pumping station utilizing submersible pumps which has a dry well.

In general, this type of pumping station is used where relatively small flows are to be handled. Flygt of Canada who have probably supplied in Ontario, more of these units than any other manufacturer, supply their sewage pumps in five sizes; 3.5, 6, 9, 18 and 25 H.P.

The 3.5, 6 and 9 H.P. units are relatively light in weight and suitable for small installations. The 18 and 25 H.P. pumps are considerably heavier and not suited to manual removal.

It is interesting to note that it is possible to increase the capacity of an existing flygt installation by converting from 3.5 H.P. to 6 H.P. This is accomplished by changing the stator and the impeller. The same pump casing is used for both sizes.

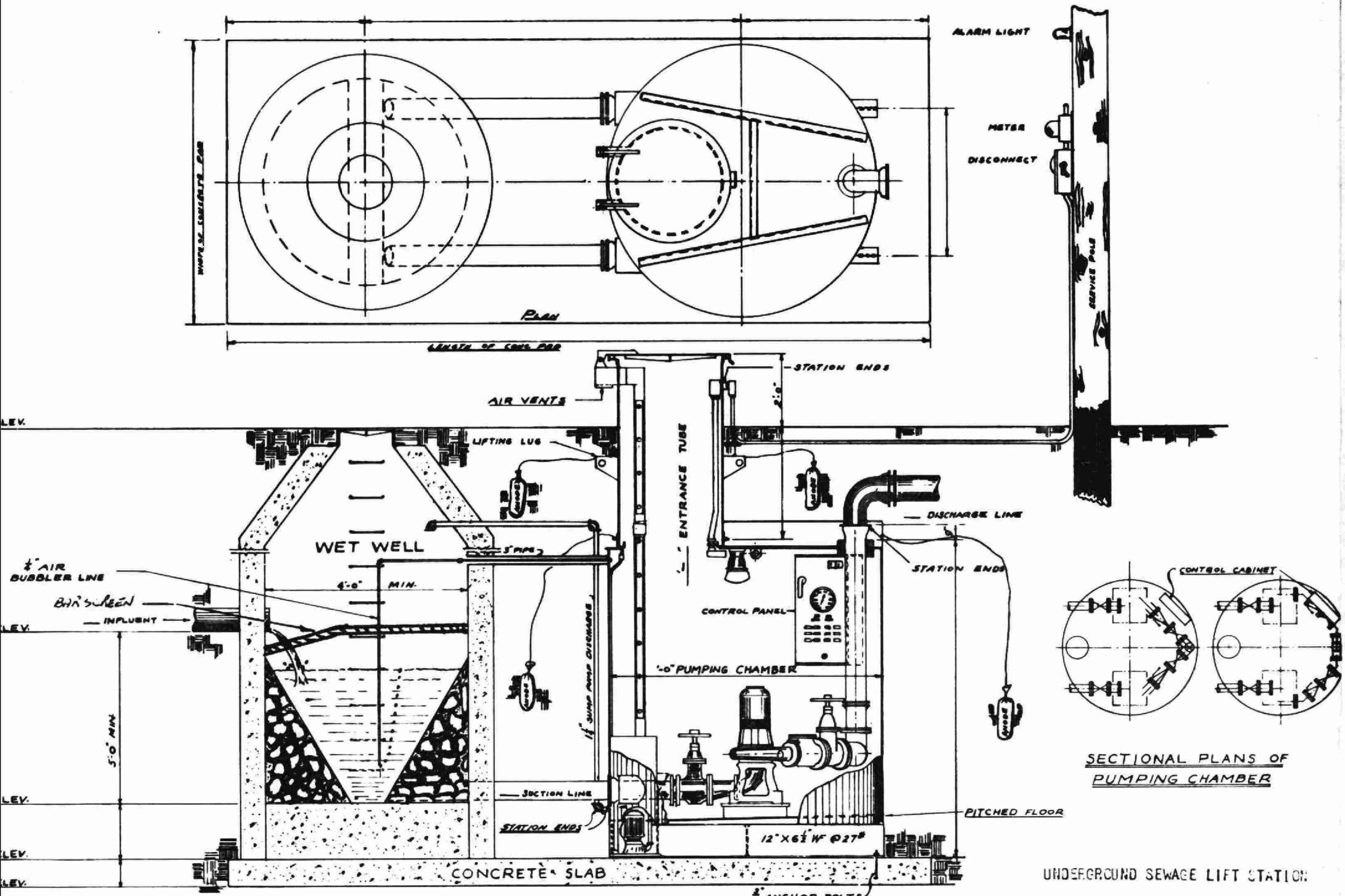
Maintenance

Maintenance should be relatively simple on this type of pumping station. This is especially true where the pumps are supplied complete with a type of automatic coupling. With this coupling, when an inspection of the pump is necessary, it is a relatively simple job to lift the pump out of the wet well. The need for dewatering is eliminated.

On a routine inspection of the pump, oil levels and quality of the oil should be checked. Insulation resistance tests should be made at routine intervals preferably every three months, to determine conditions of motor windings and electrical feed cables.

If for any reason, there is soubt as to the correctness of rotation of the pump this should be checked before placing the pump back into service. The importance of this was recently driven home when it was discovered that a submersible pump at a sewage treatment plant had been running in the wrong direction for several months. This pump had been returned to the supplier to have seals and bearings replaced. When it was returned to the plant it was placed into service with no particular thought to rotational direction. This action was prompted by the plug-in type of connection on the pump. However, the supplier had failed to note the existing electrical connection when the pump was received for repair. Upon reassembling the pump he inadvertently reversed two wires. The pump operated well enough in the reversed condition, not mind you to anticipated operating levels, but well enough to draw the conclusion that the supplier in the repair of the pump had possibly altered the clearance on the impellor, thus altering the discharge head conditions. I would re-emphasize the importance of checking the direction of rotation before lowering the pump into the wet well. This may be easily done by observing which direction the pump "kicks" on the start up. If the pump twists in a counter-clockwise direction on start up, the direction of rotation is clockwise and vice-versa.

The control of the pumps in this type of pumping station is usually accomplished by simple float switches.



—SETTING CROSS SECTION—

SECTIONAL PLANS OF PUMPING CHAMBER

UNDERGROUND SEWAGE LIFT STATION

As the sewage rises in the wet well, a series of switches are used to start and stop the pump motor. One of the most common float switches in use today is the flygt level regulator.

III. FACTORY BUILT PUMPING STATIONS UTILIZING CONVENTIONAL CENTRIFUGAL PUMPS

Until fairly recently, pumping stations handling up to 1500 gallons/minute were completely constructed in the field. These are referred to as custom built or field built stations. However, with advancing technology, it became possible to build pumping stations within the confines of a factory,shipping a finished unit to the job site. There are advantages and disadvantages to both field built and factory built stations.

Probably, the most important advantage of the factory built station is the saving in costs which are possible. Thus, it is that more and more we are finding factory built stations are replacing field built stations for flows up to 1500 GPM. (I use this as a general figure; it would be possible, I think, to get factory built stations to handle higher flows if a special request were made).

Both three pump and two pump stations are available with the latter being the most popular. Since a normal domestic house connection is 4-inches in diameter, almost all pumps supplied have a 4-inch discharge and are capable of handling solids up to 2½-inches in diameter. For flows below 100 GPM, as already mentioned, the use of 4-inch pump becomes uneconomical and thus the use of pneumatic ejectors.

PUMP MAINTENANCE

Pumps in package pumping stations are usually of the integral or close-coupled type of construction utilizing a single shaft for both pump and motor. The impellors are of the non-clog variety and in the case of a 4-inch pump are capable of passing 2½ inches dia. solid material. Most pump designers prefer a pump speed of 1150 rpm except in cases where relatively high heads are required.

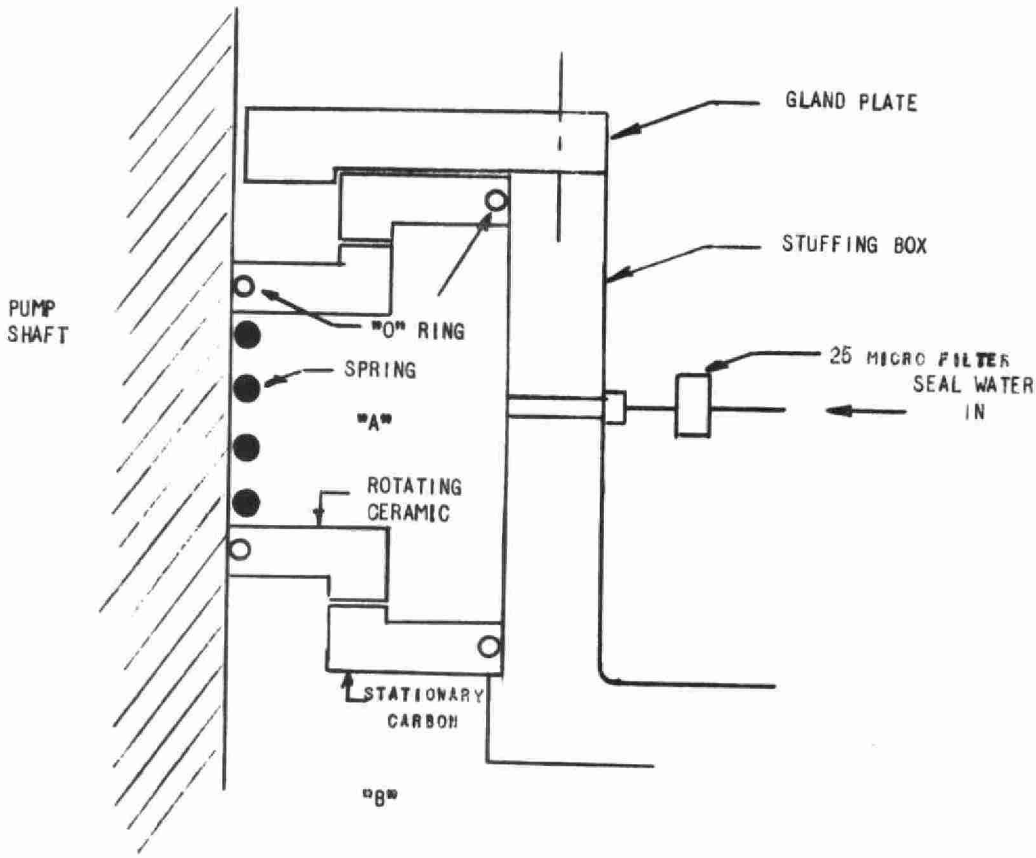
Pumps should be dismantled annually to determine the condition of the impellor, shaft, volute and wearing rings. If after several inspections, it is apparent that annual inspection is not required, the period of inspection should be extended. The items requiring the most frequent attention will be shaft sleeves if the pump is manually packed and wearing rings. The latter tend to wear more rapidly due to the nature of the material pumped.

Pump shaft sealing is accomplished through one of the following two methods:

a) Conventional Packing - This consists of a stuffing box, packing and gland plate. Sealing fluid is sometimes introduced into a seal cage or lantern ring. Since the sealing fluid generally leaks, this creates an unsanitary and odorous situation if sewage is used as the sealing fluid.

b) Mechanical Seals - These are much more common for package sewage pumping stations. The mechanical seals can take many forms but are generally double seals, consisting of two graphite rings and two ceramic rings, O rings and a spring.

One ring of each material forms a seal at the top and bottom of the seal casing. The spring which is located between the two seal rings exerts a pressure on both which should be greater than the casing pressure at shut off head. The graphite ring is stationary to the stuffing box and the ceramic ring rotates on the shaft. The seal is lubricated with a pressurized liquid from either a separate source or filtered sewage. The success of the mechanical seal depends firstly upon its lubrication and secondly on the pump being balanced and vibration free. The latter two we usually have little control over, this being primarily the manufacturers responsibility. Seal lubrication is however, something that the operator should be actively pursuing.



Referring to the drawing below, one can see the basic elements of the seal; the ceramic and carbon elements, the O rings and the spring.

Pressure exerted at "A" by the sealing fluid should be greater than pressure at point "B". As a general rule 75 percent of the discharge pressure of the pump is considered adequate for the pressure in the stuffing box. In most cases on sewage pumps, seal water is taken from the discharge side of the pump and passed through a 25 micron filter. It is extremely important that the filter on this seal water line be kept as clean as possible. If head loss reduces the pressure of seal

water below that of the pressure on the sewage side of the seal, there is a real danger that particles of grit will wash between the O ring and the shaft. This will result in wear on the shaft with possible metallizing required in a short period of time and will also result in a rapid deterioration of the seal faces as more and more grit gets into the seal housing.

One quick way of determining the condition of the filter is to open the pet-cock on the seal casing when the pump is not operating. If a continuing flow of clear water can be obtained, due to the positive suction head, the filter is clear.

It should be emphasized that the pet cock on the seal casing should never be opened while the pump is operating. This will result in an immediate drop in pressure in the seal casing and the pressure on the sewage side will be greater. This could easily result in a small amount of gritty material washing under the O-ring.

As far as cleaning the filter is concerned, either oakite or gunk (carburetor cleaner) are recommended by our mechanical maintenance section. It is not recommended that the filter be blown out with compressed air following a soaking in cleaner, as this may result in dirt and grit being forced towards the inner face of the filter insert where it may easily be washed through and into the seal casing. A good supply of spare filter inserts should be on hand at all times.

If excessive quantities of grease or some similar material make filtering the sewage and using it as seal water an undesirable proposition, fresh water sealing arrangements can be provided. These may be custom made or a stock item is available now. In any case, the most important thing to remember is that a supply of clean water under sufficient pressure (usually 75 percent of discharge pressure) is essential to the efficient operation and long life, of the seal.

MOTOR MAINTENANCE

Motors in most underground pumping stations are squirrel cage motors. Wound rotor motors are sometimes found where flow matching is required but these are, in most cases, limited to custom built pumping stations. Due to the common discharge header in package pumping stations, one motor will rotate clockwise and one will rotate counter clockwise. An arrow on the pump casing will indicate the desired direction for each motor. Maintenance of the motors is generally very simple. Many motors are shipped with custom greased bearings. In cases such as this, no lubrication is required. Where grease fittings are provided, lubrication should follow manufacturers' specifications. It is important to remember that too much greasing is often more dangerous than too little. Periodically, it may be wise to dismantle the motor and thoroughly clean the components. A resistance check of the windings may indicate that a thorough drying is necessary. Insulating varnish may be applied to cleaned surfaces.

In the case of wound rotor motors, collector rings should be kept clean. Ordinarily, the rings will require only occasional wiping with a piece of non-linting cloth. Abrasives such as emery cloth should be avoided. Brushes should move freely in their holders and make firm contact with the rings.

PANEL MAINTENANCE

1) Circuit Breakers

These are usually sealed units and maintenance is impossible. It would be advisable to check resistance across the contacts (periods of these checks determined by how frequently unit is used) to determine their condition. Measured resistance should normally be very low (1 or 2 Ohms) and this will increase as the contacts become pitted and burned through use. If in doubt as to what resistance is considered excessive, the manufacturer should be consulted.

2) Motor Starters

As a general rule, all starters for motors over

20 H.P. are usually wired for a reduced voltage start. This reduced voltage is obtained through either the use of a resistor bank or an auto transformer. A time delay device of approximately 1 second usually is sufficient before full voltage or a higher step is applied to the pump motor.

Contacts in starters are accessible and may be inspected for excessive burning and pitting. Once again, a check of resistance drop across the contacts will give an indication of their condition. Contacts improve with use (or at least so state some manufacturers). Filing and dressing of contacts in some starters is considered taboo by the manufacturer and replacement in these cases should occur only when almost all contact material is gone. Fixed and moving contacts should be replaced at this time.

3) Alternators & Alternator power pack

Spare rectifiers for the power pack and the alternator relay itself should be kept in stock. Although failure of these components is not a frequent occurrence, the cost of spares warrants their being on hand when they are needed.

LEVEL SENSING AND CONTROL DEVICES

We have looked at the types of pumping stations one might encounter, and during the course of the conversation such terms as electrodes and bubbler systems were mentioned. Let us look now at the most frequently used methods of controlling the operation of pumping equipment.

The Mercury Float Switch

This is simply a mercury switch located within a floating bulb. When the bulb is tipped on a rising sewage level, the mercury closes contacts which in turn energize the motor starter. A second switch located at a pre-determined low water level de-energizes the motor stopping the pump. If more than one pump or operating level is desired,not one but several floating mercury switches may be suspended in the wet well to call a series of pumps. This type of switch is frequently used

for a high water alarm in all types of pumping stations.

Electrode operated controls

This has already been discussed in fair detail under ejectors.

This is an all electrical device which measures liquid level changes by making electrically conductive liquids part of the control. Here the liquid becomes a current carrier and when the circuit between the voltage source and the relay is completed through probes touching the liquid surface, the relay opens or closes the circuit and operates the equipment. There are usually two probes associated with this control; one to start the equipment and one to stop the equipment.

Maintenance

On this type of control maintenance of the probe in the form of a general flushing off with clear water is usually all that is required.

Bubbler Operated Controls

Bubbler control systems operate on a back pressure principle. A regulated air flow is forced through a single pipe at a pressure just sufficient to overcome the maximum liquid level in the wet well. (This is normally set 5 to 15 psi greater than maximum pressure required to maintain a flow of air at maximum liquid level). Pressure sensors, tapped into this pipe between the air flow regulator and the end of the pipe register the variations in back pressure set up by the rising and falling liquid level. The pressure sensors convert the mechanical measurement into an electrical contact closure which in turn activates the holding coil of the motor starters. An air flow rate of from 0.5 to 2.0 cfh. or a velocity of 2.8'/sec. usually obtain the best results. (This setting is normally adjusted to give 60 to 80 bubbles/minute).

Any number of pressure sensing devices may be tapped on to the bubbler pipe to provide a wide variety

of functions (control more than one pump, high level alarm etc.)

Maintenance

1. Blow out or vacuum away dust which accumulates on pressure sensing devices, relay contacts etc., the frequency of this maintenance will depend upon operating conditions but should be done no less than once a year.

2. The Pressure tank should be drained of accumulated condensate. The intake filter on the compressor should be periodically cleaned.

4. The end of the bubbler tube should be routinely inspected to avoid a build up of grease etc.

Float Operated Controls

Float operated controls mechanically translate any given level change into some number of degrees of rotation. This is accomplished through forces developed in level rise and fall which are brought to bear on an operating shaft by a mechanical linkage. Electrical devices are mounted on the operating shaft. These electrical devices may be switches, or potentiometers, which in turn control the operation of motor starters.

STATION CLEANLINESS

A clean pumping station indicates interest on the part of the operating staff. A dirty, messy station is usually synonomous with a poorly operated station.

For the sake of a few minutes a week, keep your stations free of debris, clean and well painted. This indicates interest and pride in your work and is indicative of your general attitude towards all aspects of your job.

CONCLUSIONS

Sewage pumping stations are essential links in the pollution control process. If you cannot reliably convey the raw sewage to the treatment units, you are not doing your job.

Pumps and related equipment are like any other piece of equipment. If they are properly operated and maintained, they will give long lasting and reliable service. If neglected, they will fail.

DIGESTER GAS COLLECTION AND UTILIZATION

Ray Norton
Safety and Training Officer
Division of Plant Operations

FUNDAMENTALS OF GAS PRODUCTION

The gases produced in a digester are the product of anaerobic bacterial action during digestion of the volatile contents of sludge pumped into the digester from the primary tank.

The actual process of digestion is covered by Mr. George Trewin in his lecture titled "Digestion of Sludge", therefore, this lecture will start with the types of gases produced.

CONSTITUENTS OF GAS

The gases generated through the various stages of digestion are Hydrogen Sulfide (H_2S), Carbon Monoxide (CO_2), Methane (CH_3), Nitrogen (N_2), and Hydrogen (H_2).

Methane (CH_3) is the principle gas produced through complete digestion mixed with Carbon Monoxide (CO_2) in an approximate ratio of 70 per cent CH_3 and 25 per cent CO_2; remaining five per cent made up of N_2; H_2 and possible one to two per cent of H_2S.

The gas produced in a digester will be referred to as "digester gas" rather than sludge gas or sewer gas as it is a by-product of controlled digestion. Sludge gas and sewer gas is a natural product of decaying organic material in a sewer or sludge storage tank and is not controlled or used as fuel.

The heat value or B.T.U. content of digester gas varies from 400 B.T.U.'s to 600 to 700 B.T.U. per

cubic foot depending on various factors of sludge and digestion in comparison to 1000 B.T.U. per cubic foot for natural gas and 140,000 B.T.U. per gallon of No. 2 fuel oil.

No. 2 fuel oil can be purchased in large quantities for 14 cents per gallon, therefore, the value of sludge gas is $\frac{140,000}{14 \times 600} = 16.6$ (17) cubic feet per one cent.

During digester start-up or following periods of overloading, addition of toxic materials, or substantial temperature changes, the methane content will be reduced and the carbon dioxide will increase correspondingly and the hydrogen sulphide content may become a serious problem.

RATE OF GAS PRODUCTION

The daily rate of gas production can be expressed in the following ways:

Primary sludge: 0.5 to 1.0 cu. ft. per capita per day
or 6 to 10 cu. ft. per pound of volatile matter in the raw sludge per day
or 15 cu. ft. per pound of volatile matter destroyed per day.

Example: 10,000 persons
Suspended Solids in sewage - 200 ppm
Flow = 100 gallons per capita per day = 1 mgd

Primary plant:
50% removal of solids - ½ x 200 x 10
therefore - Pounds of solids removed
= 10 x 100 = 1000 pounds

70% volatile matter in sludge
therefore - Pounds of volatile matter added
= 700 pounds

Volatile matter is 50% destroyed during digestion
therefore - 350 pounds are destroyed.

Gas production per day:

Therefore Cu. ft. per capita = 0.5 to 1.0 x 10,000 cu. ft. = 5,000 to 10,000 cu. ft.
Cu. ft. per lb. volatile matter in raw sludge = 6 to 10 x 700 = 4,200 to 7,000 cu. ft.
Cu. ft. per lb. volatile matter destroyed = 15 x 350 = 5,250 cu. ft.

Minimum and maximum hourly rates of gas production may be from 45 to more than 200 per cent, respectively, of the annual rate of production. Continuous or intermittent mixing of digesters often aids in providing uniform gas production.

GAS COLLECTION AND STORAGE (Digesters)

There are three common types of digester roofs.

1. A fixed roof of concrete or prefab steel construction, some are domed roofs and others are flat.

   The gas storage area under a fixed roof is restricted to the total area between the maximum overflow water level and the roof, a gas dome located in the centre of the roof of approximately three to five feet high by three to six feet in diameter is the drawoff point by piping located inside the digester extending up into the gas dome.

   One of the most important aspects of gas storage under a fixed roof is the flexibility of the maximum overflow pipe or box.

It should be adjustable enough to build up the gas pressure in the digester from zero to the rated maximum pressure without changing the static level of the liquid in the tank.

2. Floating cover of prefab steel construction floats on the liquid, moves up and down with contents in upper third of tank. The area of gas storage is very limited under this type of roof, confined mainly to the centre of the roof and a gas dome, of a size similar to fixed roof types. The maximum overflow design has little or no influence on the gas storage.

3. Floating gas holding type of prefab steel construction, floats on the gas stored under it, moves up and down with contents in upper third of tank.

   This type of roof is usually on the secondary digester and will have five to eight foot skirts on the sides.

   It is domed topped but does not have a gas dome in the centre, the gas stored under it can be withdrawn from a "gas well" located anywhere in the roof. The gas storage area is equal to the diameter and 3/4 of the depth of the skirt of the roof.

   They usually require six inch W.C. to lift it from the liquid. All pressures in excess of the six inch W.C. up to rated maximum pressure say of twelve inch W.C. is stored gas, although the pressure per square inch is low and measured in inches water column the total volume of stored gas is large, up to 35,000 to 50,000 cubic feet.

SAFETY DEVICES

All digester roofs have a pressure relief valve and a vacuum relief valve, generally located on the gas dome or centre of the roof if of a gas holding type. In most instances they are of a dry weighted cup type but some are designed with a weighted diaphragm and some are of a weighted wet type using kerosene for sealing the unit.

The pressure relief valve is a safety valve. It is generally set to relieve the gas pressure under the roof at one inch W.C. but not more than two inch W.C. above the maximum pressure. The main gas line P.R.V. is set to maintain all pressure. Excess of the P.R.V. setting is passed through the valve to the waste gas burner.

The vacuum relief valve is generally rated at two inch W.C. and a vacuum created under the roof greater than two inches W.C. will cause the valve to open and allow air to enter the tank. If the valve was not there or did not operate, an excessive vacuum under the roof would cause it to collapse into the tank. This condition is more prevalent to fixed roof types during sludge withdrawal but can happen to floating covers under certain conditions. However, it is most desirable to withdraw sludge from the digester in such a way as to prevent a vacuum from drawing air into the tank as you are now creating an explosive mixture under the roof.

DIGESTER GAS PIPING

The gas collection pipe in the digester will be from two and one-half inches in diameter to four inches in diameter although the gas system piping may be larger. The intake end of the pipe will extend well above the maximum overflow level and possible scum level. The digester gas main from where it leaves the digester to the boiler contains a number of units that are required for operation and safety control.

Starting at the digester outer wall these units are; moisture trap or water accumulator, (see chart No. 1).

<u>Description</u>

Heavy steel tanks with gas proof fittings and removable top.

It is fitted with a drain pipe approximately 1/3 up from the bottom, the drain pipe contains a shut-off plug valve and at the end a small moisture trap or drip trap. The bottom 1/3 is filled with water up to the level of the drain pipe, they are sometimes fitted with a sight glass on the side.

<u>Purpose</u>

The gas passing through the tank will surge about in the tank depositing the moisture in the gas and the impurities with the moisture.

<u>Service Required</u>

Daily drainage of excessive moisture collected.

<u>GAS METER OR METERS</u> (if one is located on waste gas piping)

<u>Description</u>

Usually a large type heavy steel construction recording mechanism located top section, records gas production in cubic feet in increments of 10ths. Lower half contains bellows made of sheepskin or neoprene with centre sections made of zinc.

Purpose

To record total daily gas production, gas meter on the waste gas burner piping simplifies calculating amount of gas used by the boiler and/or wasted.

Service Required

At least twice weekly drainage of all your petcocks in the bottom of the meter. Each petcock drains a separate area.

PRESSURE REGULATING VALVE (P.R.V.)(Waste gas burner pipe)

Description

Diaphragm type, sized to suit piping, diaphragm can be sheepskin or neoprene. Controlling action can be either weights of lead rings or spring tension adjusted by a slotted plug threaded in spring housing. All adjustments whether spring or dead weight is in inches water column W.C.

Purpose

This valve is the key regulating control for the whole gas service system from digester to boiler.

Example

If it is required to have eight inches W.C. in the gas main to operate the boiler the P.R.V. located on the waste gas burner pipe is set by weight or spring tension to stay closed until the pressure of the gas main reaches eight inches W.C. All pressure in excess of eight inches W.C. will be allowed to pass through the P.R.V. to the waste gas burner. (See chart No. 2)

Service Required

The one-quarter inch plug in the bottom of the valve housing should be removed and the condensate trapped there, drained out at least once per week, more often if large amounts of water are found.

The top of the valve should be removed every three months and the diaphragm checked, if made of sheepskin a small amount of "neetsfoot" oil should be spread over the leather to keep it soft and pliable. Lift the diaphragm up and down by the centre stem to check on freedom of movement of diaphragm, stem, and valve.

Check vent in the valve housing top for obstructions. Normal life of diaphragm will be from three to five years.

FLAME ARRESTERS (See chart No. 3)

Location

In waste gas piping downstream from the P.R.V.

In the gas main at the boiler between the P.R.V. and failsafe valve.

In all pilot gas pipe referred to as a flame check.

Description

Type number 1 - square construction sized to suit piping. End plate removable, fastened with type of wing nut. Core is of box design with handle. Grid is corrugated aluminum plates.

Some square flame arresters or traps have a further protective feature such as a thermal controlled shut-off. The thermal plug will soften at 260 degrees and allows the spring loaded valve to close off the flow of gas through the unit. Time of valve action approximately 15 seconds.

Type number 2 - round construction sized to suit piping. Top of unit body removable, core is round in design, approximately two inches deep, grid is corrugated aluminum strips. Some round type units have a thermal valve plug located in the centre of the core, it will also soften at 260 degrees and allows the spring loaded valve inside the unit to close. Closing time approximately 15 seconds.

There are other protective units available such as a shut-off thermal valve (without grid). Flame checks containing small round grid.

Purpose

As the name implies a "flame arrester" or "flame trap", contains any flashback of fire from an ignition point, i.e., combustion chamber of boiler.

The small openings through the corrugated metal plates are too small to allow the passage of the ignited and expanded gases.

Service Required

The one-quarter inch drain plug in the bottom of the unit body should be removed and the condensate drained out at least once per week, more often if a large amount of water is collected. The grid should be removed every three months and cleaned and inspected, inlet and outlet ports should also be inspected for undue corrosion etc. If the unit has

a thermal valve control, careful inspection of the valve stem and spring should be made to ensure unit is free of movement and valve stem is not corroded and stuck in the open position.

Core can be soaked in kerosene then blown out with a compressed air hose.

MOISTURE OR DRIP TRAP (See chart No. 4)

Location

Connected to the gas main at the low end of a graded horizontal pipe and the bottom end of all vertical pipe and at the water accumulator.

Purpose

To trap and hold all moisture that condensates from the gas and drains to the lowest point of a given section of pipe.

Service Required (Manual type only)

Drain condensate from all drip traps daily. When draining trap, check for undue looseness of operating handle and for gas smell. If a strong or continuous gas odour is noticed coming from the trap then further maintenance is required.

DIGESTER GAS COMPRESSOR FOR GAS MIXING IN DIGESTERS

Gas will often escape past the drive shaft packing gland of gas compressors and although the escaping gas can be reduced considerably by tightening the gland nut it will be found that it cannot be stopped entirely. Further tightening would cause friction and eventually score the shaft creating even more leakage and costly repairs.

Service Required

Will be as per manufacturers maintenance specifications.

MANOMETERS

Description

There are a number of different types. Most use an oil in the graduated tubes. The pressure recorded is read in inches water column.

Other types use mercury in the tubes, pressure is read in inches mercury and a conversion table used for readings in inches water.

Example

One inch of mercury equals 13.6 inches water. Other conversion factors:

one inch of water = 0.58 ounce per square inch
27.7 inces of water = one pound per square inch (1 psi)

Purpose

To record gas pressures in the digester, service pipe and waste gas pipe.

Service Required

Checked daily for pressure readings.

The waste gas pipe meter should not show pressure, if it does then there is an obstruction in the pipe somewhere.

Check for condition of sight glass.

Check for oil level at zero readings, add, if necessary. A sudden surge of gas will blow the oil from the meters or a vacuum will draw the oil down the pipe when it returns it will be broken up in water and will have to be replaced and the tube cleaned.

Regulations governing the storage and use of digester gas as a fuel. Gas Utilization Code 166/66 In 1966 the Energy Act was revised to include the gas produced in a digester.

Bill 48 Section 1 Subsection 7

states:

"gas" means natural gas, manufactured gas or liquified petroleum gas or any mixture of any of them;

Therefore, digester gas being classified as a manufactured gas and used as a fuel will be subject to any and all regulations governing the use, storage and transmission of a manufactured gas.

PART 2 GENERAL

Section 18, subsection 4, all adjusting, purging, servicing and testing of appliances and piping systems shall be done by a certified gas fitter - Form 309

In Ontario Water Resources Commission pollution control plants this type of work is done by persons having either a gas fitters certificate - Form 309 or a gas maintenance fitters certificate - Form 312 or the work is done by others under the direction of a person having one or the other of these certificates.

PURGING OF DIGESTER GAS PIPING

Before any gas piping or appliance is removed for servicing or replacement the section containing such equipment shall be isolated from the gas supply and purged of all explosive or dangerous gas with the use of carbon dioxide (CO_2) or nitrogen (N_2) or a combination of the two. Air is not to be used.

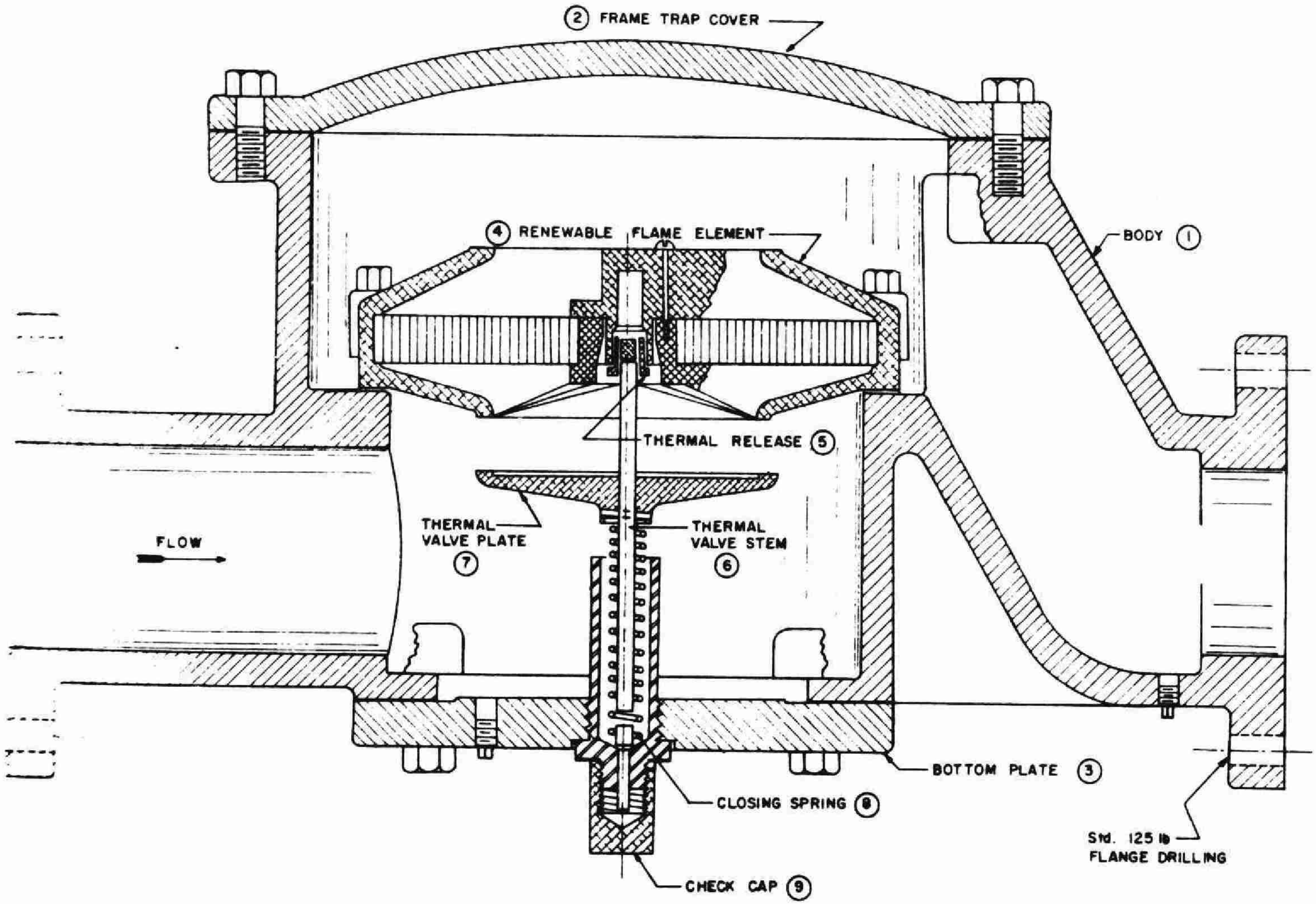
VENTING OF APPLIANCES

All control regulators, appliances, and recording manometers having vents shall be vented to the outside of the building. If a common header is used it shall not be less than three-quarters inch diameter. Plug valves shall be used in all gas piping. Plug valves shall be installed in all gas service piping to pressure regulators and manometers downstream from the connecting union.

The gas piping and appliances thereon suffer from the corrosive action of the impurities in the gas.

In some plants the gas piping has had to be replaced after 18 months to two years service. In other plants the pipe has lasted for eight to ten years.

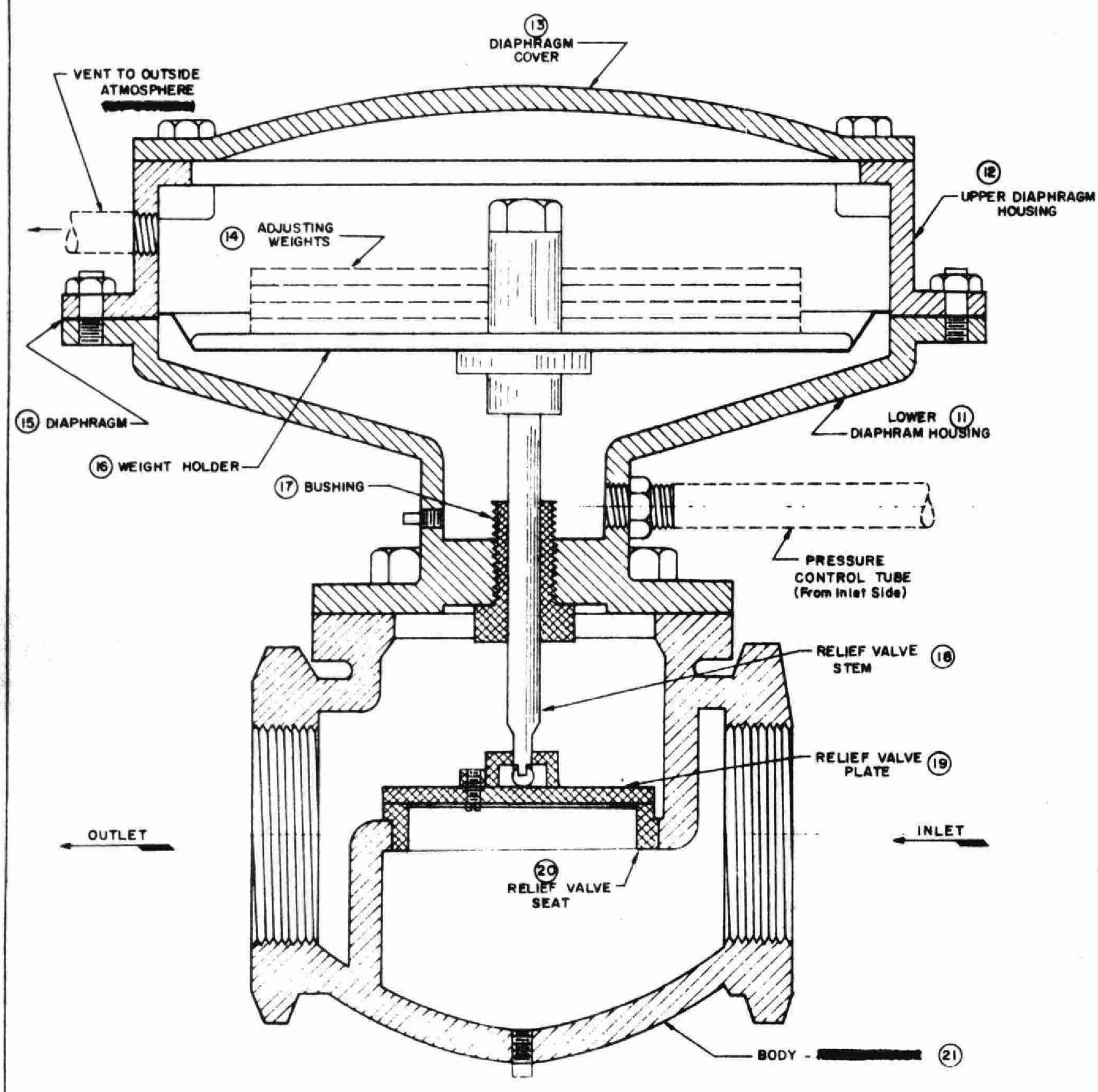
Replacement of this pipe is a difficult, dirty and sometimes dangerous job. The safety precautions and regulations should be known and observed.



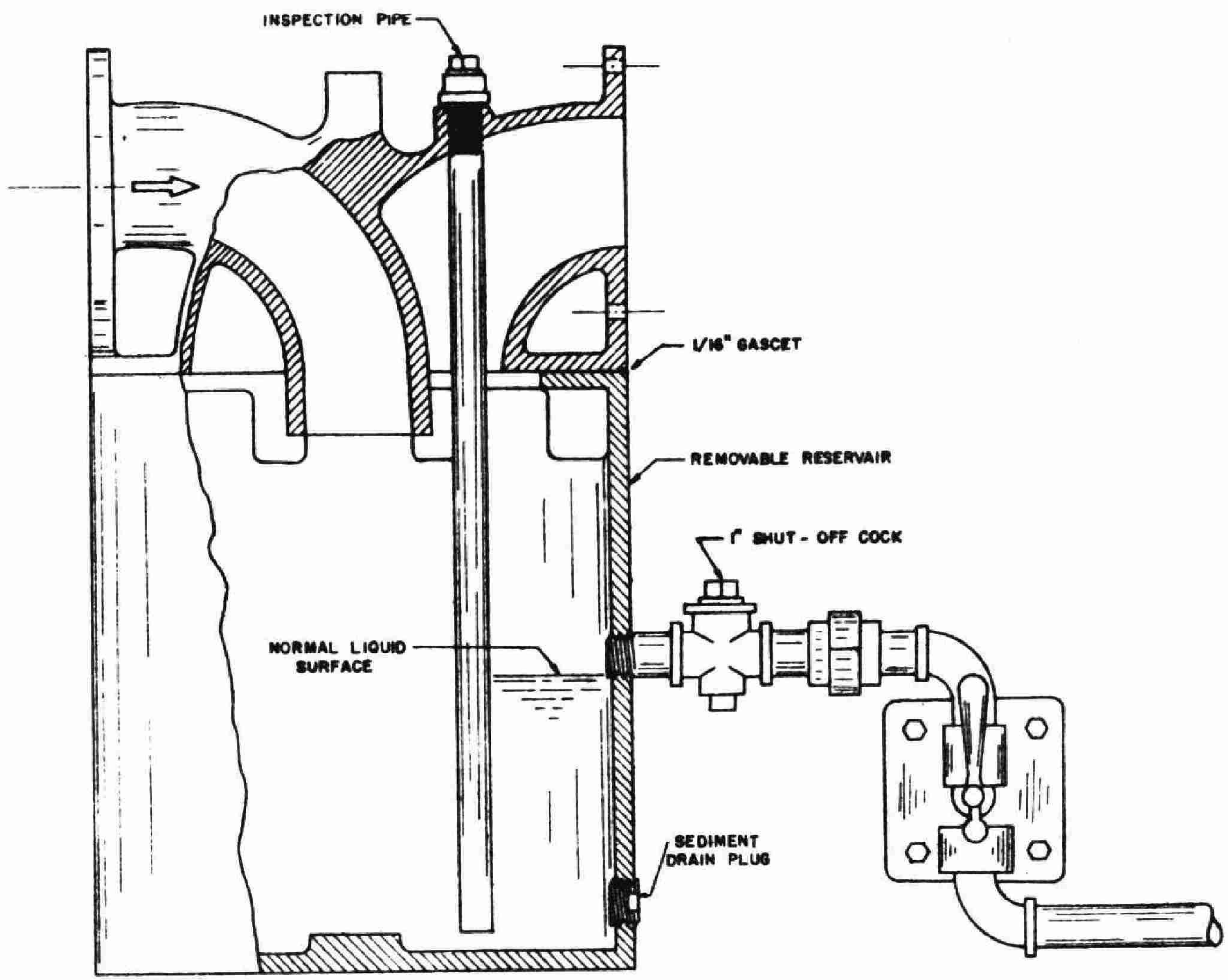
(2) FRAME TRAP COVER
(4) RENEWABLE FLAME ELEMENT
BODY (1)
THERMAL RELEASE (5)
FLOW
THERMAL VALVE PLATE (7)
THERMAL VALVE STEM (6)
BOTTOM PLATE (3)
CLOSING SPRING (8)
CHECK CAP (9)
Std. 125 lb FLANGE DRILLING

FLAME TRAP

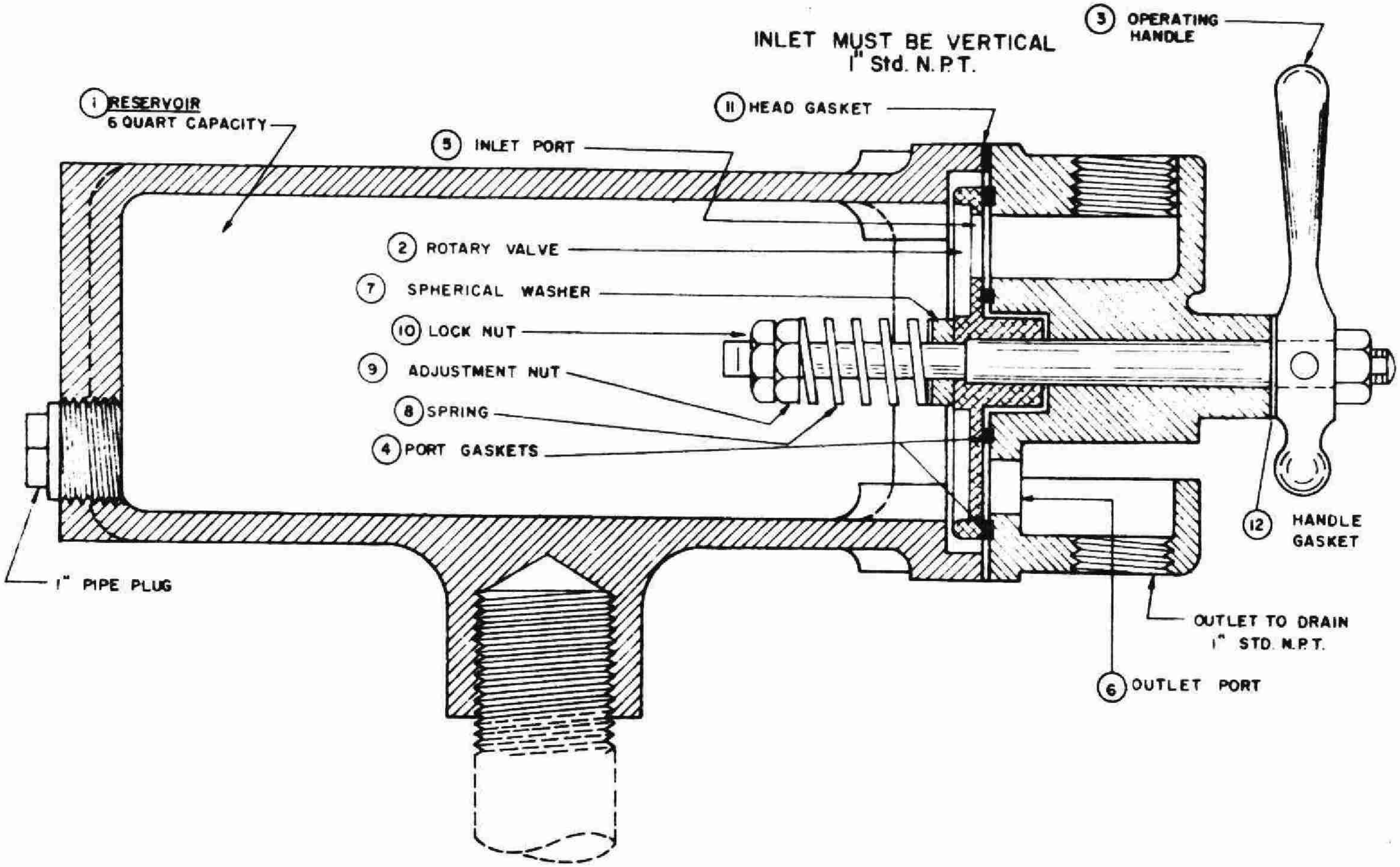
PRESSURE REGULATING VALVE



CONDENSATE AND SEDIMENT ACCUMULATOR



DRIP TRAP



INLET MUST BE VERTICAL
1" Std. N.P.T.
3 OPERATING HANDLE
1 RESERVOIR
6 QUART CAPACITY
11 HEAD GASKET
5 INLET PORT
2 ROTARY VALVE
7 SPHERICAL WASHER
10 LOCK NUT
9 ADJUSTMENT NUT
8 SPRING
4 PORT GASKETS
12 HANDLE GASKET
1" PIPE PLUG
OUTLET TO DRAIN
1" STD. N.P.T.
6 OUTLET PORT

INTERPRETATION OF ANALYTICAL RESULTS IN THE SEWAGE FIELD

C. J. Howes
Division of Research

As plant operator, you are asked to sample the incoming raw sewage and the treated effluent once or twice a month, depending on the arrangements made by your district engineer or operations engineer. As you know, the purpose of these samples is to show the overall performance of the plant and to show whether the treated effluent meets the OWRC objectives. Several weeks after sample submission, you receive a copy of the sample results sheet, which contains various numbers. Today I will try to show you what some of these numbers mean, and how they relate to the performance of your plant.

In addition to the samples you send to the OWRC laboratory, you will also take samples at the plant and analyze them yourself, for control of plant operation. From this discussion you may get a better idea, how to use the results of your tests at the plant.

Perhaps at this point I should make some remarks on the technique of sampling, because the results of any tests on a sample are only as good as the sample.

- You must always remember to rinse the sampling can, dipper (milk bottle) at least once with the liquid you are sampling, before pouring the sample in the bottle. This is especially important when you have only one sampler, and you are taking samples of raw sewage and final effluent in sequence.

- You must take the sample at a point where the liquid is thoroughly mixed; this is of special concern for suspended solids. Both points are essential, to ensure that the sample truly represents the liquid sampled at that time. Where you only take a single sample, that is called a grab sample, usually sufficient for analysis at the plant for process control.

As most of you know, grab samples are generally not acceptable for the once or twice monthly submission to the OWRC laboratory for analysis of plant performance. The reason is of course, that the quality of raw sewage and the flow both vary widely during a single day.

To overcome this, your district engineer will have asked you to take a series of samples from the same stream over eight hours at intervals of one or maybe even a half hour. In the simplest case, the hourly samples are all equal in volume. Preferably, anyway for the raw sewage and where applicable, the primary effluent, the hourly samples should be measured according to the flow at time of sampling.

Example 1

1 mgd	-	100 ml or 10 ml per 100,000 gal.
0.9 mgd	-	90 ml
1.2 mgd	-	120 ml

In special cases, an automatic sampler will be used to obtain a 24-hour sample; ideally this equipment should take the sample proportional to the flow.

If you will refer to the appended charts, you will find one showing a block diagram of an aerobic system. Along with this will be a chart showing the tests which would normally be done on these samples. The second block diagram is of the anaerobic portion

of the system and the chart showing the usual tests done on these samples.

These charts and diagrams should be used whenever samples are sent in, in order to have the proper tests done.

Most of the results from these tests will be considered in this talk.

BIOCHEMICAL OXYGEN DEMAND TEST (BOD)

The first laboratory result to be discussed is the BOD_5 or the 5-day Biochemical Oxygen Demand. This test is used to determine the oxygen requirement of sewage. The BOD result indicates how much oxygen is required to provide purification by the aerobic action of bacteria metabolizing the organic fraction of the sewage and by the protozoa which feed on both the organic constituents and on the bacteria.

The result of the BOD test, as an indicator of the organic component of sewage, is reported in milligrams per litre. This data, along with the flow can be used to calculate the organic load that will be imposed on the aeration system; it can also be used to calculate the efficiency of removal of organic material in the primary clarifier and the overall BOD removal in the plant. Knowing the amount of oxygen required to oxidize a given amount of BOD, it is then possible to determine whether or not there is sufficient air being delivered to accomplish this.

Example 2

When the following data is known, these typical calculations can be made.

BOD in the raw sewage	=	325 mg/l
Primary effluent BOD	=	165 mg/l
Flow	=	1,250,000 gpd
BOD in final effluent	=	13.0 mg/l

Find - 1. the BOD applied to the aeration tanks in lb/day

e.g. 165 x 10 x 1.25 = 2,060 lb/day

2. the per cent BOD reduction through the plant

e.g. $\frac{325-13}{325} \times 100 = 96.0$ per cent

3. BOD reduction in the primary clarifiers

e.g. $\frac{325-165}{325} \times 100 = 49.2$ per cent

SOLIDS DETERMINATIONS

Total solids are determined by weighing an aliquot of sludge in a tared dish or pipetting a known volume of sewage into a tared dish and evaporating it to dryness at 103°C. Total solids are required as a prerequisite for determining the volatile content of a material or as one step in producing a suspended solids figure by difference. In sludges the total solids are used to calculate the amount of food being fed to a digester or the amount of material leaving a digester. These are two of the factors required to determine the digester efficiency. This will be considered later.

The dissolved solids figure on an analytical result sheet gives little operating information to an operator. Values for dissolved solids will represent, in many cases, such soluble fractions as hardness, chlorides, etc. present in the town water supply.

An analytical result sheet usually shows three columns for solids. These are total, suspended and dissolved solids. The centre column, or suspended solids, carries the significant figure for sewage plant operators.

Suspended solids are determined by one of three methods.

1. By the Millipore Filter

2. By the Gooch Crucible method
3. By the filter paper method.

In all three the principle is the same. A known volume of the sample is filtered through a medium which is dense enough to retain all the particles which are in suspension. The filter media is then dried and the amount of solids determined by weight difference. This test may be used for determining the amount of solids fed to the digester, primary clarifier solids removal efficiency and in addition suspended solids in the aeration tanks and return sludge.

Suspended solids in the raw sewage is not used to any great extent but if a plant does not have a means of collecting a good raw sludge sample, then a 24-hour composite of the raw sewage and primary effluent can provide a means of determining the loading to a digester.

Example 3

Solids in Comp. of raw sewage = 325 mg/1
Solids in Comp. of primary effluent = 215 mg/1
Daily flow - 2,350,000 gpd
then (325-215) x 2.35 x 10 = 2,580 lb. solids/day.

If the volatile fraction is 70 per cent, then the volatile solids applied to the digester would be 2,580 x 0.7 = 1,810 lb. per day.

The calculation to show the per cent efficiency across the primary clarifiers is as follows:

$$\frac{\text{solids in primary effluent} = 215 \text{ mg/1}}{\text{solids in raw sewage} = 325 \text{ mg/1}}$$

per cent solids retained in clarifier =

$$325 - \frac{215}{325} \times 100 = 33.8 \text{ per cent.}$$

Primary settling will reduce the suspended solids by 40 per cent to 60 per cent and the removal is quite often better than the per cent BOD removal.

The suspended solids in the mixed liquor is used to determine the F/M ratio and it may be used to control the aeration section (e.g. to increase or decrease the amount of DO being carried) and also can be used to determine the amount of activated sludge which can be wasted.

Example 4

The MLSS is 3,000 mg/1 and should be 2,000 mg/1
The aeration tank capacity is 300,000 gallons
Return sludge = 7,000 mg/1

The problem is, "how many gallons should be wasted to reduce the mixed liquor suspended solids the desired amount?".

First, determine the number of pounds of solids which would have to be removed to reduce the suspended solids by 1,000 mg/1.

This would be 3,000 - 2,000 = 1,000 mg/1

$$\text{and } 1{,}000 \times 10 \times \frac{300{,}000}{10^6}$$

$$\text{or } 1{,}000 \times 10 \times .3 = 3{,}000 \text{ lb.}$$

With return sludge solids of 7,000 mg/1, how many gallons would have to be wasted?

$$\frac{3{,}000 \text{ lb.}}{7{,}000 \text{ lb}/100{,}000 \text{ gal.}} = \frac{3{,}000}{7{,}000} \times 100{,}000 =$$

$$= 43{,}000 \text{ gal.}$$

$$\text{or } \frac{3{,}000 \text{ lb}}{.07 \text{ lb./gal.}} = \frac{3{,}000}{.07} = 43{,}000 \text{ gal.}$$

If the pump used for wasting is rated at 300 gpm, the required wasting time would be $\frac{43,000}{300}$ = 143 minutes or 2 hours and 23 minutes.

ANALYTICAL RESULTS FROM ANAEROBIC SECTION

Plants equipped with a primary clarifier must have some means of disposing of the raw sludge. This could be either by anaerobic decomposition in a digester, aerobic decomposition in an aerobic digester, or by vacuum filtration.

Example 5

Given a total solids in the raw sludge of 45,000 mg/l (4.5%), raw sludge pump capacity = 50 gpm.

Raw sludge pump cycle six times daily for 12 minutes per cycle.

The raw sludge feed to the digester can be calculated from results reported either as mg/l or per cent solids. With solids reported as mg/l, the amount of sludge pumped would be:

50 gpm x 12 min x 6 cycles = 3,600 gal/day
or .0036 mgd.

Solids loading is then .0036 x 10 x 45,000 = 1,620 lb/day.

If the solids are reported as a per cent dry solids, then the loading is $3,600 \times 10 \times \frac{4.5}{100} = 1,620$ lb/day.

Example 6

Suppose the flow is 1.2 mgd.
suspended solids in the raw sewage 284 mg/l
suspended solids in the primary effluent 99 mg/l
the reduction in the primaries is 284 - 99 = 185 mg/l
which is 185 (mg/l) x 10 lb/gal x 1.2 mgd =
185 x 10 x 1.2 = 2,220 lb/day.

This, then, is the amount of sludge to be applied to the digester or filter. Frequently it is the volatile fraction that we are interested in.

Suppose the volatile fraction is 73.6 per cent, then 2,220 x .736 = 1,634 lb. volatile matter are applied to the digester each day.

This is the part of the raw sludge that we are most concerned with and this figure can be used to determine the amount of gas that generated in an anaerobic system. It can also be used to calculate the overall efficiency or amount of digestion.

The digester efficient (the amount of digestion) may be calculated using the formula - -

$$P = \left[1 - \frac{(100-R)D}{(100-D)R}\right] 100$$

This is a simple calculation where --

P = per cent reduction of volatile matter,
R = per cent volatile solids in the raw sludge, and,
D = per cent volatile solids in the digested sludge.

Example 7

Given R = 73.6 per cent
D = 49.8 per cent

then,

$$P = \left[1 - \frac{(100-73.5)49.8}{(100-49.8)73.6}\right] 100$$

$$= \left[1 - \frac{26.4 \times 49.8}{50.2 \times 73.6}\right] 100$$

$$= \left[1 - \frac{1,315}{3,695}\right] 100$$

$$= \left[1 - .3530\right] 100$$

$$= .647 \times 100$$

$$= 64.7 \text{ per cent}$$

If the digester efficiency is known, gas production may be determined as one pound of volatile matter destroyed will usually yield 12 cubic feet of gas. This gas will be approximately 65 per cent methane, and will deliver about 650 BTU/cu.ft. If you know your BTU consumption you can readily calculate the length of time you should be able to run on gas or oil.

Other tests which will be used to effect digester control will be volatile acids, pH and alkalinity.

The volatile acids will normally run from 100 to 500 mg/l. If they increase rapidly and are accompanied by a decrease in alkalinity, then the digester may be in trouble.

The following may be described as being satisfactory conditions.

1. Methane content of gas between 55 per cent and 75 per cent and the total of methane and carbon dioxide 95 per cent of gas produced. Plant operators cannot measure this.

2. Digester sludge total solids below 15 per cent with an optimum of eight per cent and a volatile content of 50 per cent.

3. pH between 6.8 - 7.4
at pH 6.5 to 6.0 digestion is inhibited
at pH 4.5 to 4.0 effective digestion ceases.

4. volatile acids below 2,000 mg/1
normal about 300 mg/1.

5. alkalinity above 2,500 mg/1.

These figures are optimum and slight variations will not be significant but should be watched.

GENERAL

Additional analyses may, from time to time, be requested by the operations engineer or the district engineer. These will undoubtedly be to determine some of the components due to industrial wastes. Tests occasionally may be required of the following: nitrogen, chemical oxygen demand (COD), alkyl-benzyl-sulphonate (ABS), phenols, and heavy metals such as copper, chromium, lead, zinc. All these tests may have a bearing on the operation of a plant but must be regarded as "specials".

IN-PLANT ANALYTICAL TESTS

The minimum laboratory procedures required at all plants are the half hour settling test and dissolved oxygen determinations. The half hour settling test is used to control MLSS. The normal operating range would be from 150 to 300 ml of sludge after one half hour of settling. If, your settled volume increases without an increase in solids then you have a bulking problem. This test is a simple means of controlling the amount of sludge being wasted.

With the half hour test and suspended solids results, the Mohlman Index (SVI) may be worked out.

Example 8

1/2 hr. settling test - 15% or 150 ml
suspended solids = 2,000 mg/1

$$\text{the SVI} = \frac{150}{2000} \times 1{,}000$$

$$= 75.$$

The SVI is significant only within the plant where it is produced. Generally an SVI of 150 or less means good settling and thus trouble free operation.

For dissolved oxygen determinations the Winkler method is to be preferred, if a meter is not available, and in doing mixed liquors, the copper sulphate sulphamic acid modification must be used. The Miller method should be used only as a last resort when no other means of determining a reliable DO is available. With copper sulphate sulphamic acid, the chemicals inhibit the action of the biota and aid in the coagulation of the sludge thus assisting in the sedimentation.

DO should normally run between 2 - 4 mg/1. If the value drops below 2 mg/1 or goes above 4 mg/1, it does not mean that the process is failing but that some attention should be paid to the aeration system to find out why there has been a change. Within limits, this could be due to nothing more than wasting excessively to give a high DO because of low solids or a rapid build up of solids to decrease the DO.

Finally, the last point to consider is the chlorine residual. This data is not used to calculate any part of the process efficiency but is an indication of the sterility of the sewage effluent. Providing a Cl_2 residual of 0.5 mg/1 is maintained after a minimum contact time of 15 minutes, the effluent should be able to meet OWRC objectives for coliform counts.

TABLE 2

ROUTINE ANALYSES REQUIRED IN ANAEROBIC SYSTEMS

SAMPLE	No.	SOLIDS TOTAL	SOLIDS VOLATILE	VOLATILE ACIDS	ALKALINITY	pH
RAW SLUDGE	7	X	X			X
SUPERNATANT LIQUOR	8	X	X	X	X	X
SLUDGE TRANSFER	9	X	X			X
DIGESTED SLUDGE	10	X	X			X

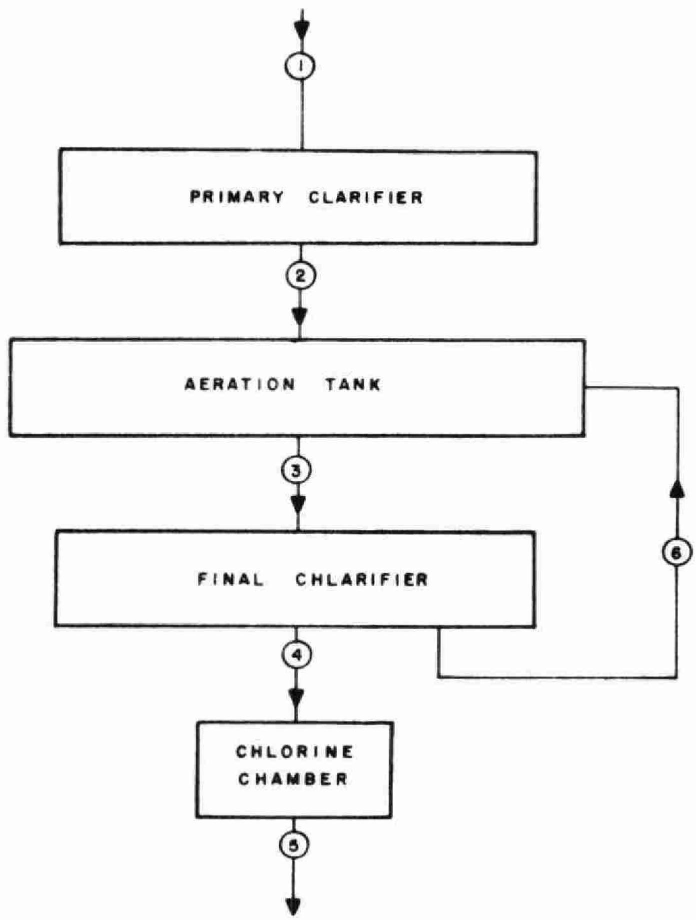
TABLE I

ROUTINE ANALYSES REQUIRED IN AEROBIC SYSTEMS

SAMPLE	No.	B.O.D.	SOLIDS				Cl_2 RESID.	pH
			TOTAL	SUSPENDED	DISSOLVED	VOLATILE SUSP.		
RAW SEWAGE	1	X	X	X	X	X		X
PRIMARY EFFLUENT	2	X	X	X	X			X
MIXED LIQUOR	3			X		X		X
FINAL EFFLUENT	4	X	X	X	X			X
CHLORINATED EFFLT.	5						X	
RETURN SLUDGE	6			X		X		X

FIGURE No. 1

SAMPLING LOCATIONS WITHIN AEROBIC SECTION



1 - RAW SEWAGE

2 - PRIMARY EFFLUENT

3 - MIXED LIQUOR

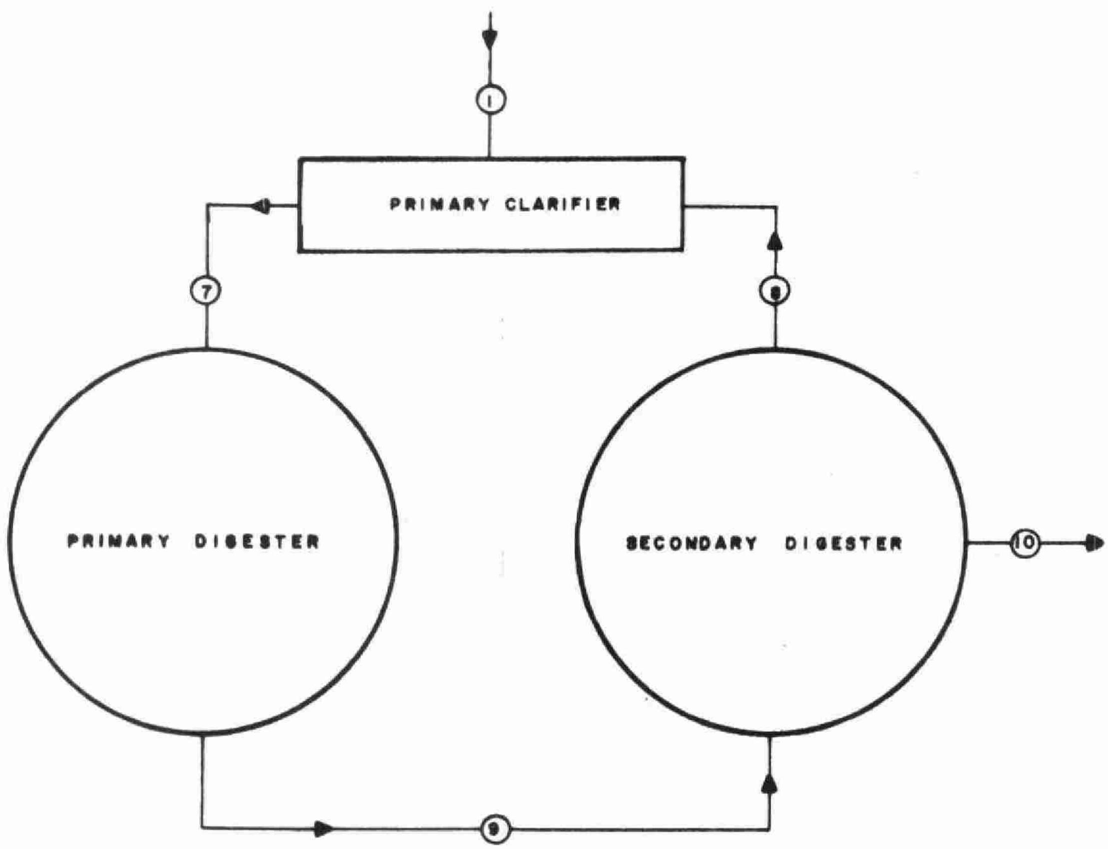
4 - FINAL EFFLUENT (INFLUENT TO CHLORINE CONTACT CHAMBER)

5 - CHLORINATED EFFLUENT (TO RECEIVING WATER)

6 - RETURN SLUDGE

FIGURE No. 2

SAMPLING POINTS WITHIN ANAEROBIC SECTION



1 – RAW SEWAGE

7 – RAW SLUDGE

8 – SUPERNATANT LIQUOR

9 – SLUDGE TRANSFER

10 – DIGESTED SLUDGE

MODIFICATIONS OF THE ACTIVATED SLUDGE PROCESS

G. H. Kay

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INTRODUCTION

Although the activated sludge process has proven itself to be adequate in many installations, some locations at some times have experienced disappointing results due to many causes such as, excessive peaking of the sewage flow, hydraulic and organic overloading, changing sewage composition including toxic materials, unbalanced nutrients, inadequate aeration capacities etc. These often manifest themselves in bulking sludge conditions in the final clarifier, with associated effluent quality deterioration.

Some limitations of the process are considered to be:

1. High initial oxygen demand of the mixed liquor.
2. High air requirements.
3. High solids loading on final clarifiers.
4. Tendency to produce bulking sludge.
5. High sludge recirculation ratios required for high capital investments.
6. B.O.D. loadings limited to about 35 lb/1000 cu. ft. This requires relatively long retention times, yielding high capital investments.
7. Inability to produce an intermediate quality of effluent.

To overcome these problems and to continue to improve the basic process, much experimentation has been accomplished, particularly by plant operators, and innumerable modifications have evolved. It is not the intent to catalogue all these in this lecture, but only to discuss some of the more widely accepted modifications of the conventional process.

GENERAL

The activated sludge process and its modifications have time-solids relations or stated otherwise, within limits, if the active solids in the process are increased, the time required for purification is reduced proportionately, and vice versa. To reduce the size and therefore the cost of capital expenditures for plants, the tendency has been to carry as high a concentration of aerator solids as possible. This procedure has limitations due to present-day oxygenation and sedimentation processes. Temperature also affects the process, but generally this aspect has not received as extensive attention to date.

The oxygen demand of the mass of return activated sludge is at a relatively low level, but as the fresh colloidal, dissolved, and finely divided solids of the sewage is attached to the floc, biological activity is stimulated and the oxygen demand thereof is immediately dramatically increased. The inlet demands can approximate four to five times those at the outlet end.

It is difficult to obtain a true value of dissolved oxygen values at the inlet and for this reason, these values are determined at the outlet end where adequate values are maintained to avoid zero concentrations at the inlet end.

TAPERED AERATION

Since the diffused aeration devices would ordinarily be spaced for equal supply, D.O. concentrations could become zero at the inlet end of the tank. This would result in the activity of the bacteria being retarded. A retardation of bacterial activity would necessitate longer aeration times and thereby larger units, or poor, (high) S.V.I. values, settling characteristics and effluent qualities will occur. To combat this, aeration devices are located sometimes in varied concentrations along the aeration tank length, with the maximum concentrations at the inlet end. This "modification" is termed Tapered Aeration. This allows the oxygen supply to approximate more closely the oxygen demand to avoid retardation of the bacteria's activity, yielding better treatment results and lower aeration costs.

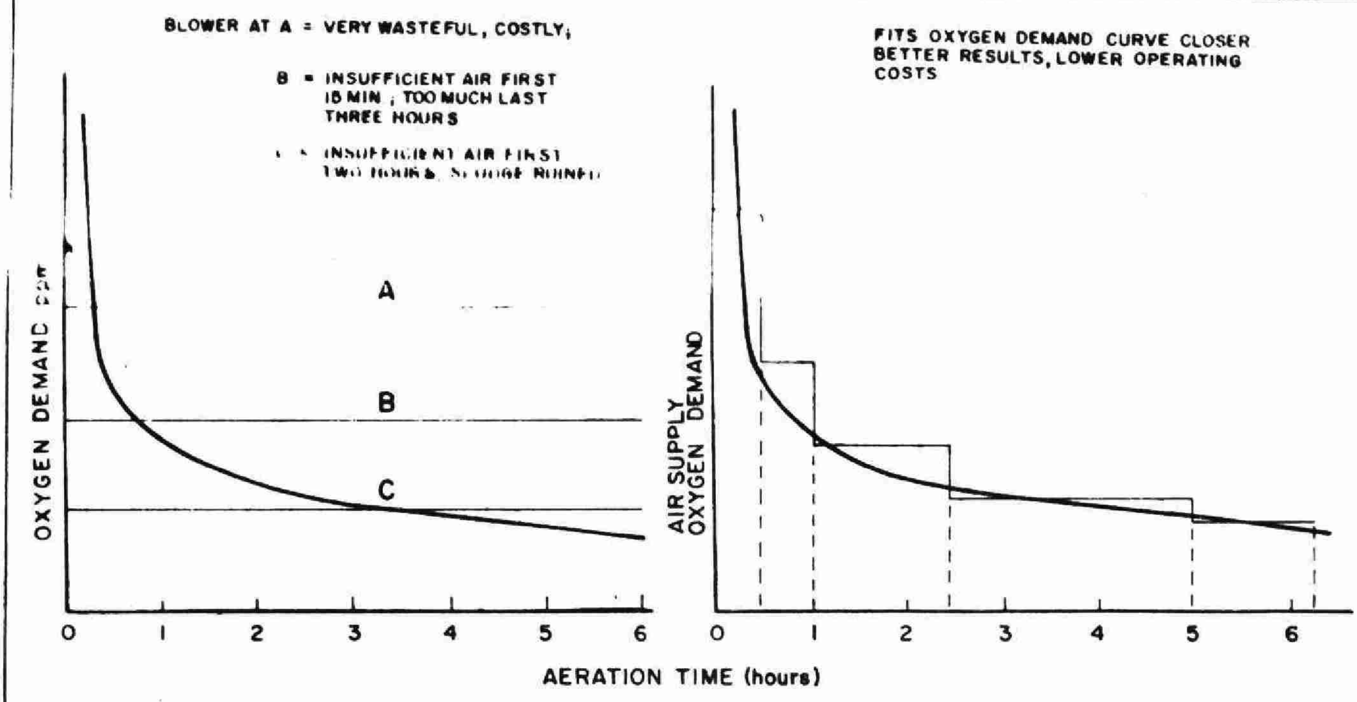
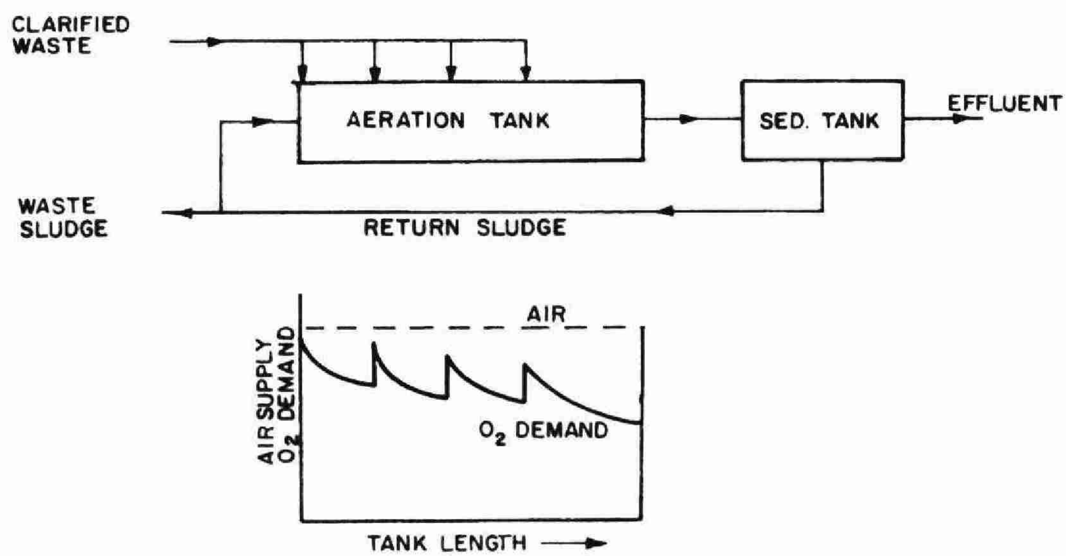


FIG. 1 TAPERED VS. CONVENTIONAL AERATION



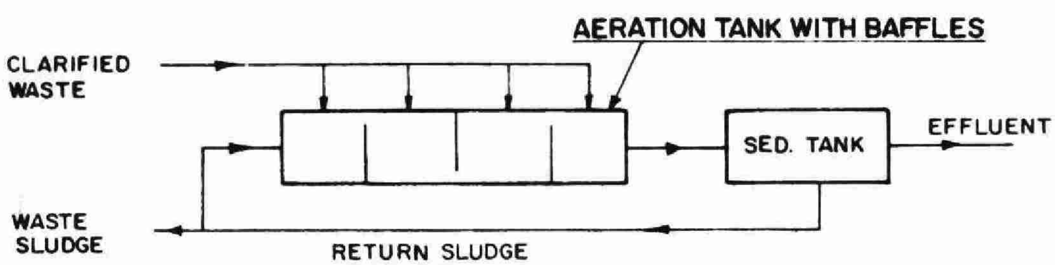


FIG. 2 STEP AERATION

STEP AERATION

This process has the sewage added to the return activated sludge rather than the reverse, as in the conventional process. This is done at many points on the combined length of the aeration tank. This too allows the initial high oxygen demand to be more evenly distributed. Around-the-end baffles are usually required to avoid short circuiting.

The name "step aeration" suggests that the process is set up only for a purpose similar to tapered aeration. Its other attributes will be discussed further, later in this paper.

As stated above, the conventional and modified processes have time-solids concepts. Basically this suggests that if proper F/M ratios are maintained, greater concentrations of solids will result in lower required total aeration times. This is also limited somewhat by the settling tank capacities. In the step aeration modification, adding sewage in increments allows a return sludge with higher solids concentration since these will be reduced to acceptable limits by dilution, before discharge to the settling tank.

DISCUSSION OF THE ACTIVATED SLUDGE PROCESS AND ITS UNITS

The basic unit of the activated sludge process is an aeration vessel. Primary settling tanks allow the removal of the larger solids of the sewage that can be easily removed by sedimentation for disposal elsewhere, and so lower the load imposed on the secondary units that are necessary for conversion and removal of the non-settleable material. In this lecture, we are concerning ourselves with the secondary treatment units. Suffice to say that secondary units may or may not be preceded by primary units, and the load on the secondary units will reflect this.

The aeration tank provides the aerobic environment for adequate acclimatized sludge added to the sewage, to rapidly clarify it by absorbing the suspended and colloidal organic material and by absorbing the dissolved nutrients thereof.

This desired result, which often can be completed in approximately one half hour, would be difficult and more expensive by any other means. The clarified liquid portion now usually containing less than 10% of the raw sewage's B.O.D. and suspended solids contents, could be discharged to a watercourse.

The sludge in its present state now contains, the solids removed from the sewage, in or on the surface of the active organisms in its zoogleal mass. This material, together with the cell material of these organisms themselves, is highly putrescible. It is necessary therefore, to ensure that this material is not discharged to the receiving waters. To do this requires separation of these solids, from the liquid to be discharged.

If the aeration process would be interrupted and a quiescent period established in the aerator, the solids would settle, and the inoffensive liquid could be discharged as the aerator effluent.

Below the liquid, there would then be a putrescible mass on the bottom of the tank representing the original activated sludge mass plus the sewage solids.

If this is to be a continuous activated sludge process, a portion of this sludge must be mixed continually with the incoming sewage. However, these quiescent periods in the aeration tank to allow sludge separation, are not practicable. Therefore, a separation, sedimentation, or settling tank is provided to complement the aeration tank. This allows the sludge to be settled and returned for continuous mixing with the incoming sewage, without the need of aeration interruption to provide quiescent periods in the aeration tank.

If aeration is provided for only the thirty minutes required to provide clarification, the sludge with its absorbed and absorbed sewage solids, is not in a condition to clarify the new incoming sewage and will require a conditioning period under aerobic conditions to improve its clarifying and settling potential. In the conventional process, this is achieved by providing this extra retention time in the aeration tank prior to any final sedimentation. Here the sludge undergoes biological oxidation for approximately six hours longer, in the liquids that will later constitute the plant effluent.

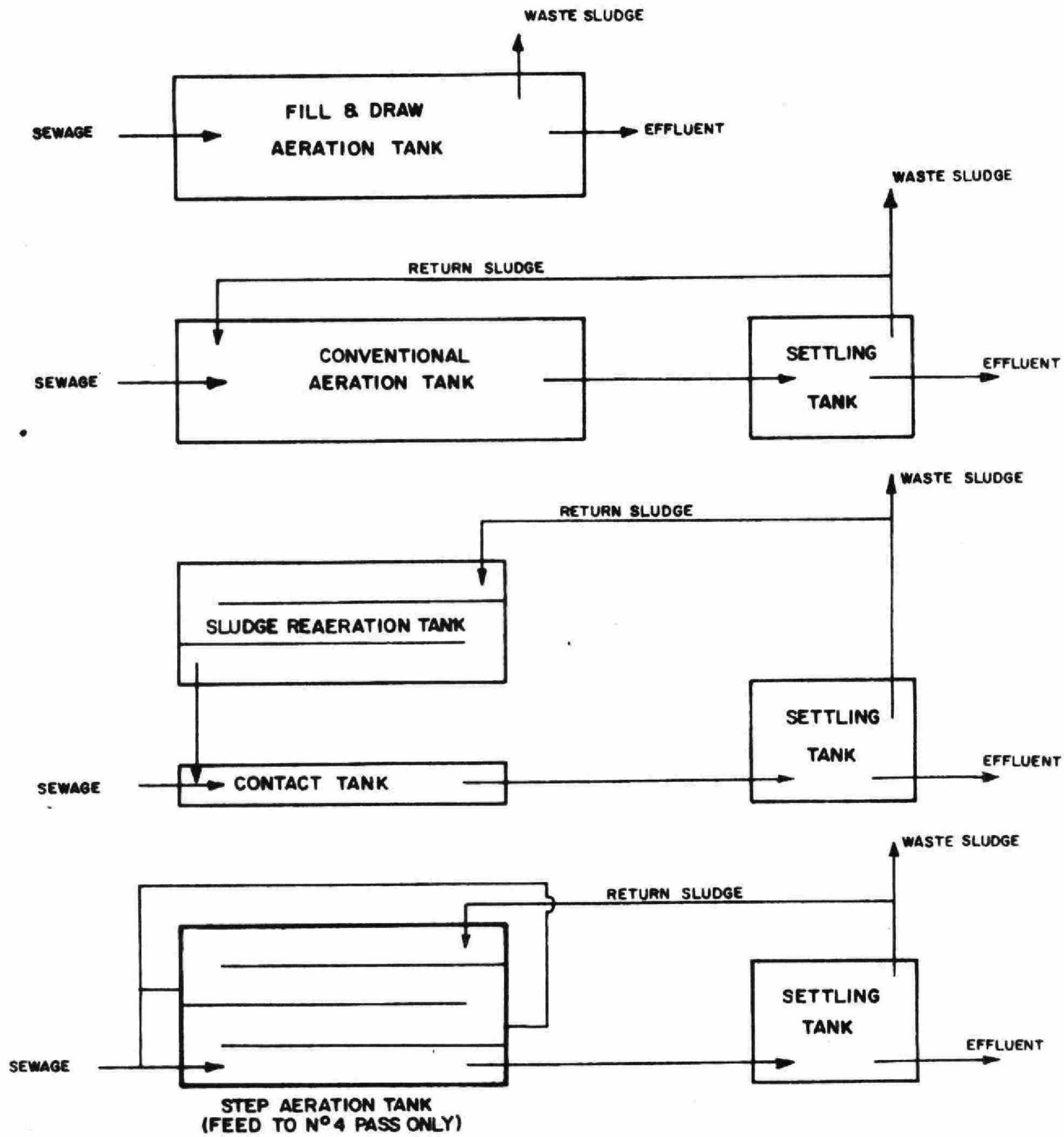


FIG. 3 SCHEMATIC DEVELOPMENT OF SLUDGE REAERATION MODIFICATION DEVELOPMENT, INCLUDING STEP AERATION.

SLUDGE REAERATION

This aeration period however, is needed for the purpose of conditioning only the sludge, not the effluent liquids, and this fact was used in the development of some modifications of the process.

In these, a new tank for conditioning the sludge alone, usually called a sludge reaeration tank, is provided on the sludge return line between the final settling tank and the head of the original aeration tank. Here the sludge that has been in aerated contact with the sewage for 30 minutes and then collected in the final settling tank, (after separation by sedimentation from the future plant effluent), is aerated alone, often for the common six hours of the conventional process.

Since this sludge usually amounts to less than 40% of the total sewage flow, this tank can be substantially smaller than the original aeration tank. Also, as previously stated, the aeration time required to allow settling and clarification, is approximately only 30 minutes. Therefore, the size of the tank originally known as the aeration tank, is reduced appreciably to supply only this 30 minutes retention time and this tank is now know as the contact tank.

This modification was almost prophesied by Arden and Lockett in their 1914 Summary and Conclusions when they said that while oxidation of the solids in the sludge proceeds, the sludge need not be in contact with the sewage.

Therefore, it can be seen that the required total aeration capacity, (in contact tank and sludge reaeration tank), can be reduced from the conventional process aeration capacity, while producing equivalent process efficiencies.

It should be noted here that the conventional continuous process plant consists of at least an aeration tank, final settling tank and an activated sludge return mechanism. This new modification called Sludge Reaeration, splits the aeration unit into two tanks, yielding three units usually called the contact tank, final settling tank and sludge reaeration tank, together with a sludge return mechanism. It is considered by some, that this process can effect a saving of 20% in the total aeration unit capacity required.

Sludge reaeration usually permits the use of a lower solids concentration in the mixed liquor of the contact tank than would be required in the conventional plant, to produce the same degree of purification in the same period of time. Therefore, it also permits, lower rates of sludge return, lower settling tank loading and possibly higher settling tank efficiencies.

Times required usually vary from 30-90 minutes in the contact tank, and five to seven hours in the sludge reaeration tank. Large quantities of active solids are maintained in the reaeration tank, out of contact with the changing incoming sewage, and therefore the process is considered to be less subject to shock loadings.

Prior to 1938, approximately only 10% of the process' total aeration capacity was utilized for sludge reaeration. Since that time, new flow diagrams have yielded reaeration percentages of total aeration capacities in the ranges of 50-67, 80-86 and 68-70, and so allowed greater acceptance of the modification.

BIOSORPTION PROCESS

Whereas the Biosorption process may not have been the first major application of the above modification, the work by Ullrich and Smith at the Austin, Texas Sewage treatment plant gave a well published insight into the possibilities thereof.

As with many developments, this modification was necessitated by operating problems at the existing plant. The sewage at Austin was always difficult to treat, with frequent sludge bulking probably due to periodic overaeration, so that during the period 1948-1951, experiments were performed, resulting in complete plant conversion to Biosorption, (extended reaeration) procedures, in 1954. Here, basically by increasing only the number of clarifiers, and converting the aeration tank to suit sludge reaeration techniques, the capacity of the plant was increased from six to 14-16 M.G.D.

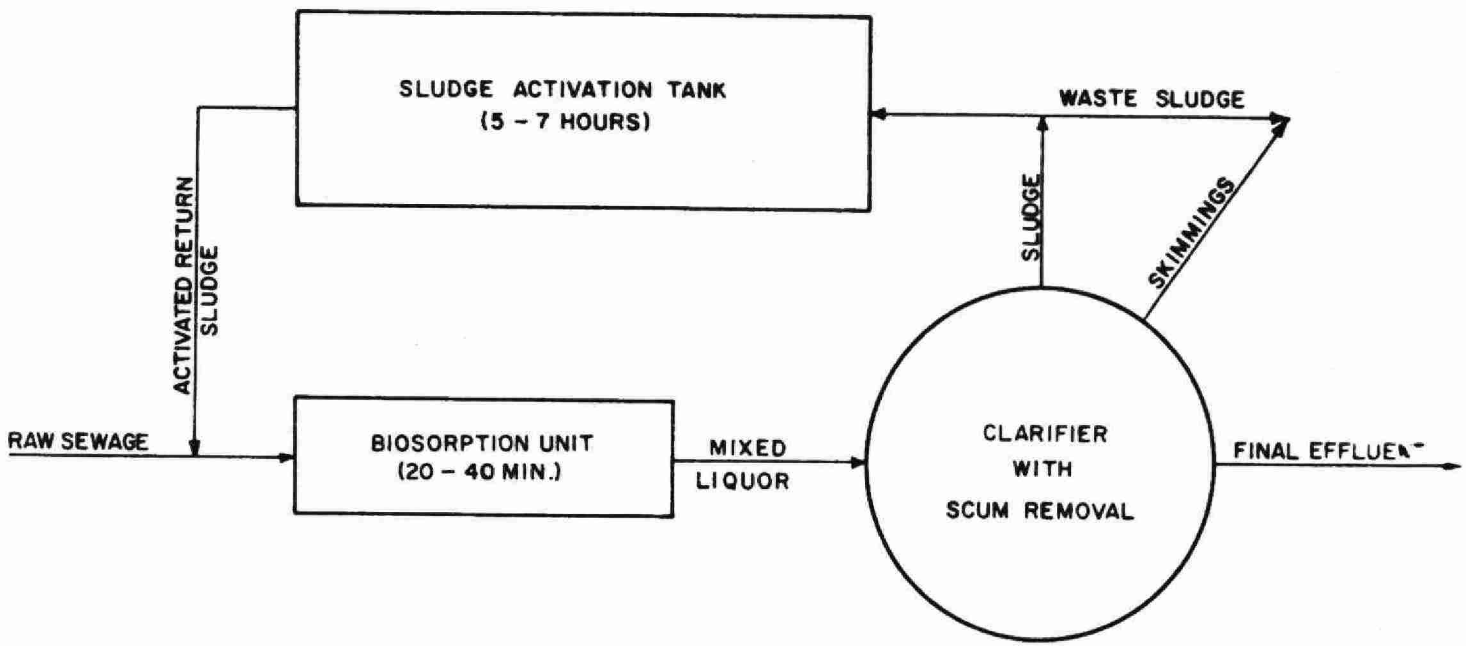


FIG. 4 SCHEMATIC DIAGRAM OF BIOSORPTION PROCESS

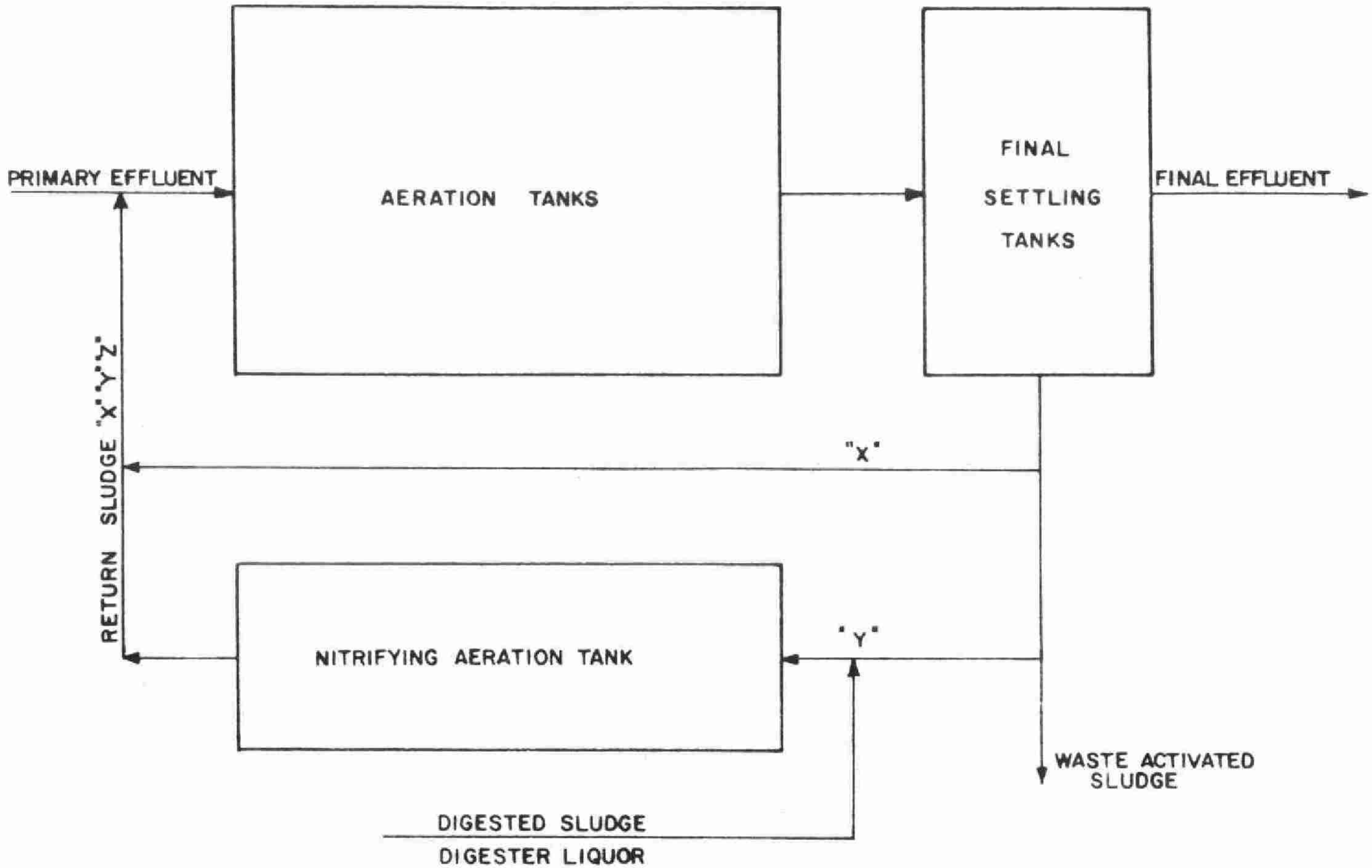


FIG. 5 FLOW DIAGRAM OF KRAUS MODIFICATION

At this plant there are no primary settling facilities. In most applications of the sludge reaeration modification however, these units are usually included.

In this modification, activated sludge approximating 40% of the total flow, that has been aerated or stabilized for five to seven hours, is added to the sewage in the contact tank. Contact time is 20-40 minutes. Clarifier retention time is usually one to three hours. Waste sludge is withdrawn from the final clarifier or from the aerator. Since this sludge contains a high energy or food level due to a high volatile solids content, a dispersed or fluffy growth yielding waste sludge problems, would be anticipated. In fact, the sludge does not compact readily and thickening units are required to obtain concentrations of 2.5% solids. This poses unusual problems in sludge disposal and lagooning of sludge is practised at Austin. Some newer application of this process have utilized other methods of sludge disposal including aerobic digestion.

However, it is claimed by Ullrich and Smith, that the process is less tempermental, less susceptible to shock loads and requires approximately only one half of the usual aeration capacity.

THE KRAUS PROCESS

L. S. Kraus, chemist of the Greater Peoria Sanitary District, evolved a modification using sludge reaeration with further refinements. The Peoria plant had been designed as a conventional activated sludge plant, but bulking of the sludge regularly produced poor effluent qualities. This was in part due to shock loads which approached values of three times design flow and these contained high-strength packing house wastes.

It would appear that these variances upset even the new sludge reaeration modification here, by yielding continued bulking in the final settling tank.

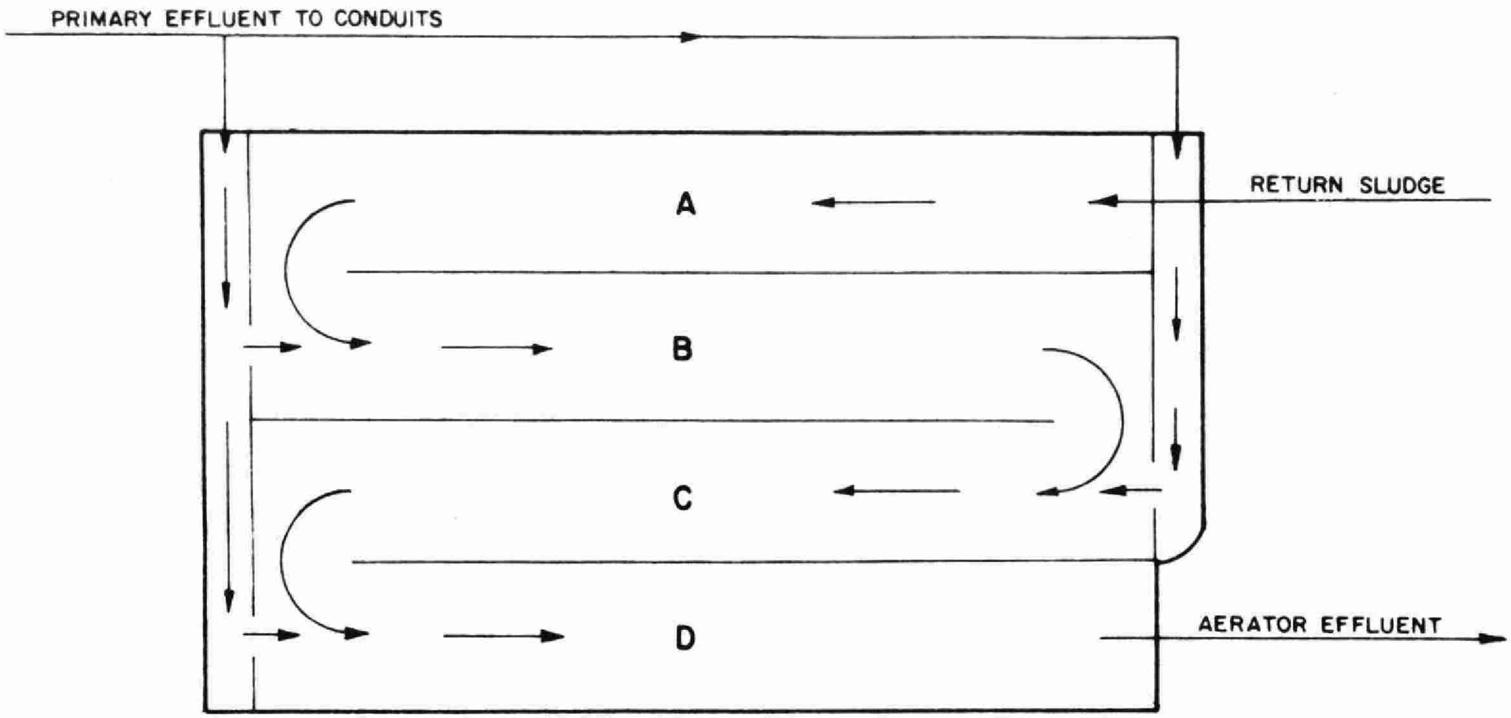


FIG. 6 STEP AERATION PROCESS EMPLOYING A FOUR PASS TANK WITH ADDITION OF SEWAGE IN B, C AND D PASSES.

As we have discussed, a low S.V.I. or a high S.D.I. usually indicates a sludge having good settling characteristics. Therefore, since the S.V.I. is equal to

$$\frac{\text{30 Min. settling test (\%)} \times 10{,}000}{\text{M.L.S.S. (p.p.m.)}}$$

Kraus chose to add some heavy solids to the aeration units to increase the M.L.S.S. content and possibly reduce the 30 minute settling test values, to lower the S.V.I. values, and so reduce the incidence of bulking. He did this by adding digested sludge and digester supernatant to the sludge reaeration tank, or as Kraus called it, the nitrifying aeration tank, to provide these heavy solids. The induced settling action of these additives could have been produced by adding other inert material such as clay, diatomaceous earth etc. However, this tank is probably the best location to discharge the supernatant and it is felt that these highly nitrified solids tend to reduce the upset caused by shock loads, in part by providing nitrates to aid in satisfying the high initial oxygen demand. Also, the aerated digester solids are considered to be converted rapidly to a heavy activated sludge.

Kraus was required to initiate a different aeration technique to provide the increased oxygenation required by the solids in the reaeration tank. This constituted two sets of aerators, being on the same or opposite sides of the tank, at different levels and so was termed Dual Aeration. This requires air to be supplied at two different pressures, due to the different depths of entry.

STEP AERATION

This process is also called "Distributed Loading", "Multiple Port Dosing", or "Incremental Feeding."

These names probably describe the process in all its aspects more properly than the title "Step Aeration." As shown in drawing number six, the procedure is to cause the return sludge to flow through the aerator with incremental additions of sewage throughout its length. As well as

leveling the oxygen demands, for equivalent average aerator solids concentrations, higher solids concentrations are possible in the initial passes for accelerated stabilizing action, with low concentrations entering the final settling tank due to dilution by the sewage. This tends to allow improved settling efficiencies.

Note also, that the original pass has no sewage added and therfore acts as a sludge conditioning tank with high solids concentrations.

Consider the situation if all the sewage is added to the last pass or channel of the aeration tank. The system then becomes a true sludge reaeration system having contact tank to stabilization tank capacity ratios of one to three or one to four depending on the number of passes.

This process is also very flexible, a quality that is extremely desirable in any secondary type sewage treatment plant due to continually changing sewage loadings etc.

The regular sludge reaeration plant has its contact tank and sludge reaeration tank capacities fixed, so that at design flow, the retention times therein, are also fixed. Since the sewage flow rates or strength may change, during the day, seasonally and over the years, the retention times in these units may change and so become inadequate, with potential deterioration of process characteristics. Only the alterations provided the conventional process, such as sludge return rate alterations and thereby M.L.S.S. concentrations alterations, are available to overcome these difficulties.

However, in the Step Aeration process, with decreasing S.D.I. values, the point of addition of sewage can be changed to only the last pass or passes of the aerator. This decreases the contact period, decreases the amount of active solids going to the settling tank and therefore, with the same rate of sludge return, increases the solids carried in the reaeration portion of the tank. The increased reaeration time will allow the solids to be properly conditioned for later addition to the contact portion of the tank.

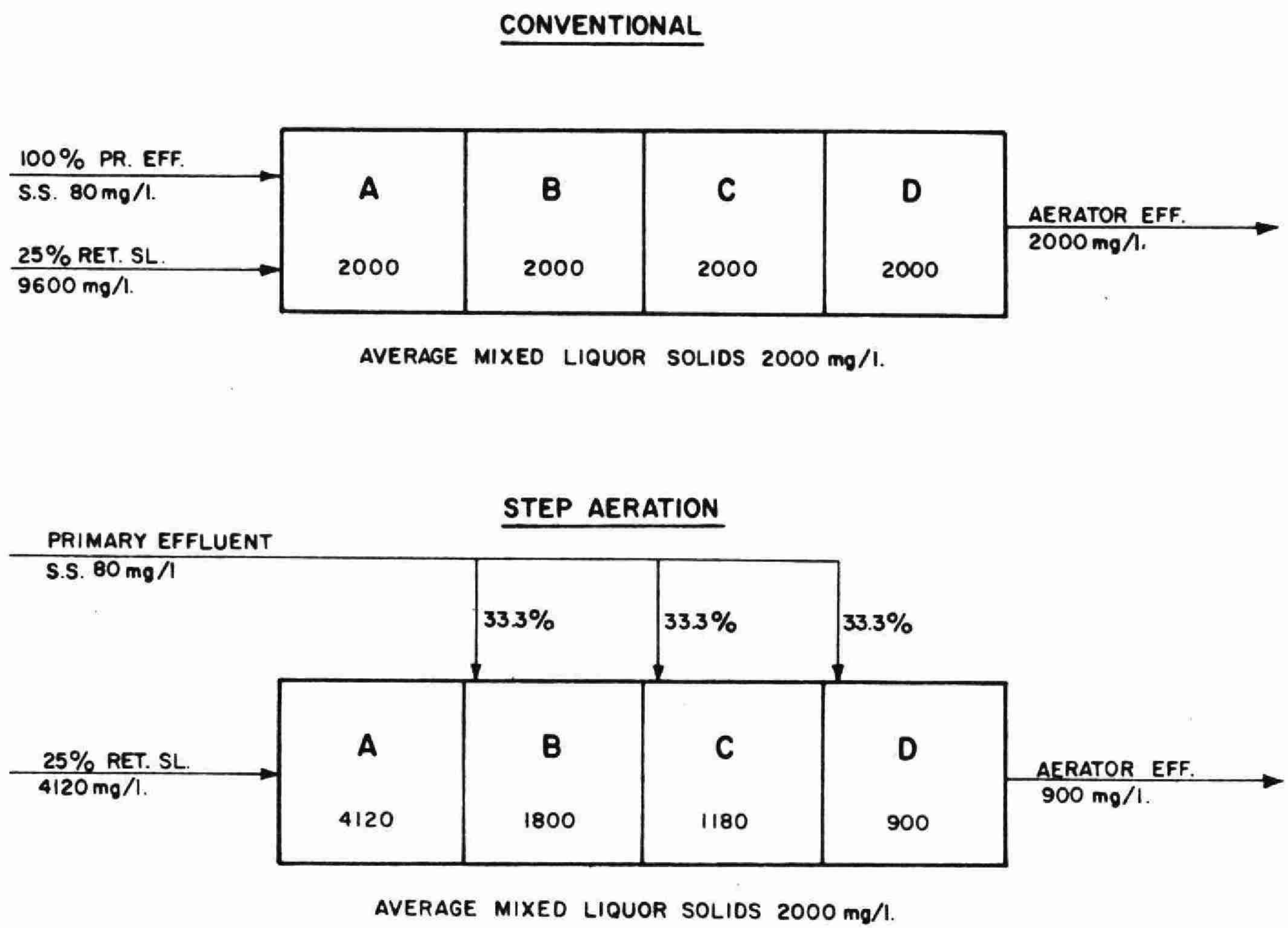


FIG. 7 COMPARISON OF SOLIDS IN RETURN SLUDGE AND AERATION TANK EFFLUENT FOR A GIVEN PLANT OPERATING ON CONVENTIONAL AND STEP AERATION SYSTEMS

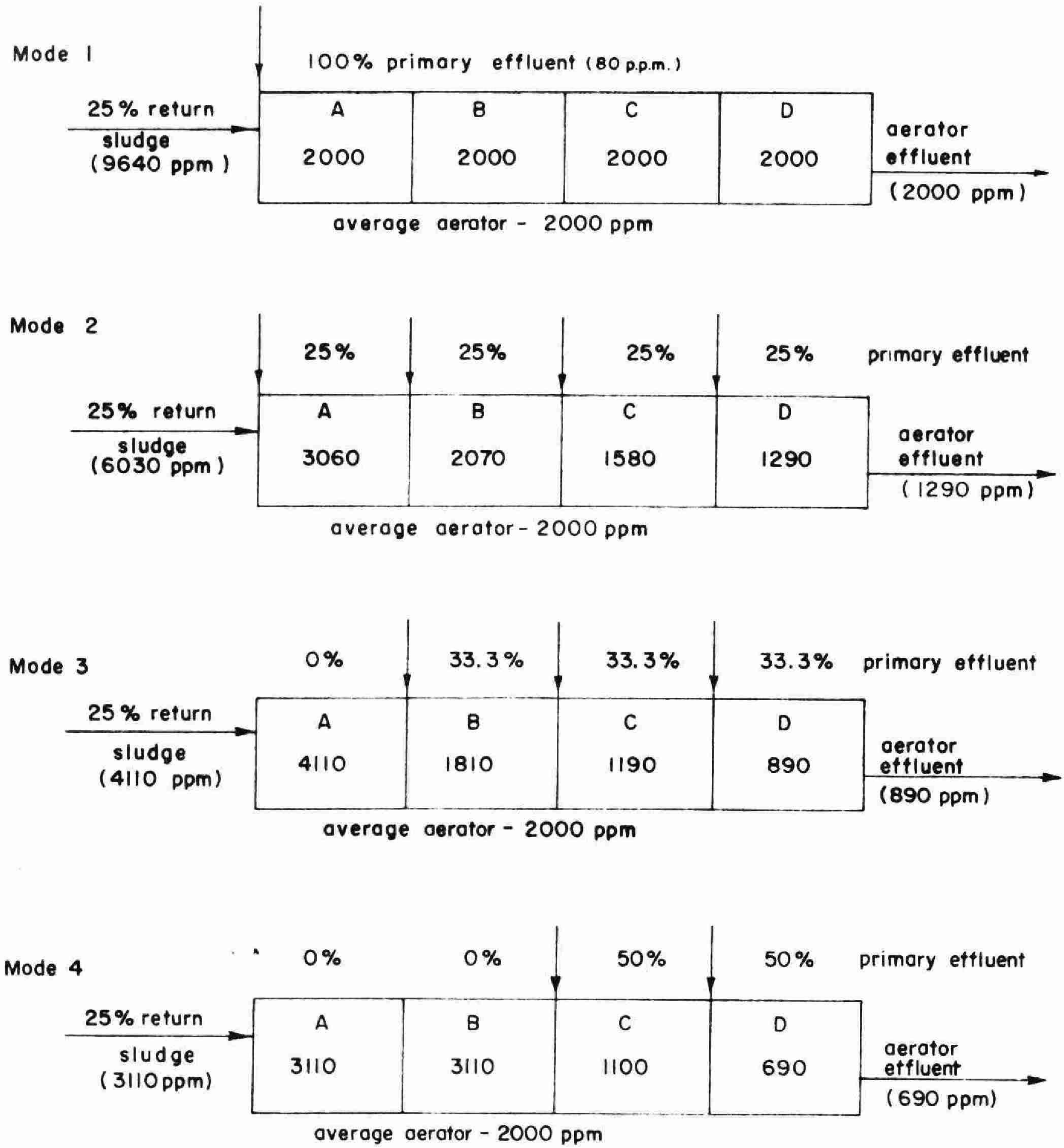


FIGURE 8

STEP AERATION

INFLUENCE OF DIFFERENT MODES OF ADDITION ON AERATOR CONCENTRATIONS AT CONSTANT AGE

Then as the S.D.I. increases, the process can be slowly reverted to ordinary sewage increment distribution to all passes or channels, and so a safety factor is provided once more, against a similar future plant upset.

In a conventional plant when the S.D.I. is decreasing, the sludge now being more voluminous, would occupy more space in the final settling tank. The required increased sludge return rates would lower the aerator solids concentration and the retention time in the aerator. We have said that the process has a time-solids relation; since both of these are being reduced, further aggravation of the sludge quality would occur. Conditions would continue to worsen until either the incoming sewage rate or strength would be reduced naturally or through by-passing.

The advantages of the step-aeration process then, are the ability:

1. To distribute the sewage load effectively throughout the aeration tank.

2. To regenerate the sludge more effectively by maintaining higher concentrations of solids in the inlet end than in the effluent end.

3. To circulate a relatively smaller amount of sludge solids throughout the final settling tanks.

The advantage of being able to vary the average concentration of suspended solids in the aerator and thus the sludge age or loading rate, by shifting the points of application of sewage, is very important, adding extreme flexibility to the operation of the plant. It allows the operation at a greater sludge age.

Loadings of 55-84 lb. of B.O.D./1000 cu. ft. of aerator are common: the conventional plant loading is 25-30 lb/1000 cu. ft.

It can be operated with no reaeration or with 25, 50 or 75% of tank volume being used for reaeration (in a four-pass unit.)

Return rates are usually 28-34%. It is the intent to pass the least quantity of activated sludge to the final clarifier, while maintaining adequate sludge age and thereby a proper F/M ratio. Average aerator solids are usually double those of the conventional plant, so that aeration time becomes 2.4 - 4.7 hours and the aerator unit can thereby be smaller in size.

SUMMARY

The main Sludge Reaeration modifications are known as:

1. Extended Reaeration
2. The Hatfield Process
3. The Kraus Interchange Process
4. Step Aeration

wherein Extended Reaeration includes those processes known as Biosorption, Contact Stabilization and the Ridgewood Biological Coagulation Process.

Total Oxidation (Extended Aeration)

In the conventional activated sludge process and most of its modifications, the average F/M ratios, using the volatile M.L.S.S. (V.S.S.) usually are kept at no less than approximately 0.3-0.35 and the Gould sludge age at no more than approximately 3.5-4.0 days, to maintain the sludge in an active state. That is, not allowing it to remain in the death phase, where endogenous respiration predominates, for excessive periods of time. This results in a conversion of sewage to sludge cell material approximating .5 lb. per lb. of 5 Day B.O.D. removed. The sludge in excess of the return sludge required, must be removed, to maintain these ratios. This is often treated in separate units being anaerobic or aerobic digesters, since the material is putrescible and cannot be otherwise easily stored.

In the total oxidation or extended aeration process, the F/M ratio is kept at such a low level (approximately 0.05 based on volatile M.L.S.S.), as to ensure that all the sludge spends a prolonged period in the death phase (endogenous respiration phase). This produces an older, less active sludge, but a great number of organisms will be present. It also ensures that the sludge will be self-consuming to produce a minimum of stabilized sludge for disposal either in the effluent or by periodic removal. Primary settling tanks are not commonly used. This then, allows the anaerobic digester with its associated operating problems to be unnecessary and makes the process attractive to installations where these units are not desirable or adequate supervision is unattainable. The extended oxygen demand of the larger colony of organisms aerobically digesting these excess solids, increases aeration costs so that the plants are more commonly found in the smaller sizes.

Care is required in translating existing parameters for sludge age and F/M ratio from the conventional process, to the extended aeration modification. It becomes more critical to know the active microbial portion of the volatile content of the M.L.S.S. This may be only 10% of these volatile suspended solids (V.S.S.) and is used by some researchers in obtaining F/M ratios.

The retention time of the sludge at equilibrium as opposed to the Gould sludge age, is related to the fraction of the sludge wasted each day and can be calculated by dividing the M.L.S.S. by the quotient of the effluent suspended solids concentration divided by the total retention time of the raw wastes in the entire system. The retention period for an extended aeration system with 24 hours aeration and four hours sedimentation would be calculated on the basis of 28 hours total retention. If the effluent averaged 28 mg/l total suspended solids and the M.L.S.S. averaged 2000 mg/l total suspended solids, the average sludge retention period would be calculated as follows:

$$t = \frac{2000}{\frac{28}{28}} = 2000 \text{ hours}$$

It can be seen that care is required in using conventional activated sludge operating parameters for evaluating extended aeration plants.

M.L.S.S. ranges are usually good - 6000 - 8000 p.p.m. and the process is capable of reductions of at least 80-90% B.O.D. and 70-80% suspended solids.

The plants are usually designed to provide 24 hours aeration rather than the usual six, with four hours settling capacity rather than the usual two of the conventional process. These values are chosen primarily to ensure adequate treatment of peak flows. The aeration time can be reduced by carrying higher M.L.S.S. concentrations, if oxygenation, settling, and sludge returning capacities can always be sufficient. In small plants, to avoid septicity or dentrification in the final tank with such high solids concentrations, sludge return rates of 100% have been usual, 200-300% are common and values of 400-500% may emerge. If dentrification occurs it will result in rising sludge conditions.

The high M.L.S.S. concentrations carried in this modification, usually ensures good B.O.D. reduction, but if severe solids build-up is allowed, in present day settling units, some of the solids tend to float out in the effluent to increase its suspended solids content. Ratios of B.O.D. to suspended solids of approximately four to one therein, are common. S.V.I. values generally range from 40 to 80.

It can be seen then that the settling tank is the key to acceptable effluents from this process, and dynamic removal such as centrifuging, rather than gravity-induced settling, may ultimately be required.

The title "Total Oxidation" is misleading since there is a portion of the bacterium's cellular capsule, apparently composed of polysacharides, which is resistant to decomposition and therefore, unless adequate sludge wastage is allowed and achieved in the effluent, periodic positive removal of solids is required. However, this is

a relatively stable material and can be discharged to an open storage tank if necessary, for subsequent disposal. High quality effluents can be produced only with separate sludge wasting at periodic intervals. It is preferable to waste an appreciable percentage (20-30%, maximum 50%) of the M.L.S.S. at each wasting, to ensure proper removal of the inert material content.

Then, since there is not complete or total oxidation of the sewage solids, the name "Total Oxidation" is a misnomer. The term "Maximum Active Solids" (M.A.S.) or more preferably "Extended Aeration" would be more acceptable.

It may be required to revise the B.O.D. analysis for effluent samples from these plants due to the effect of nitrification reactions in association with the carbonaceous B.O.D. which may cause the process' efficiency to appear lower.

AERATED LAGOONS

Sewage lagoons or oxidation ponds have been receiving a good deal of interest. Where acceptable land is economically available, these provide a unit having very low operating costs. The Commission has been using a design criteria of one acre per 100 persons, with liquid depths of three to five feet. Here algae and surface aeration, provide the required dissolved oxygen together with little associated mixing and long detention times. Recently, to reduce the area requiring for new installations or to avoid enlarging existing units, mechanical aeration units have been added to the lagoon. The lagoon then resembles somewhat, an activated sludge plant within an earthen pit. Eckenfelder has suggested that four to seven days retention time is required. If a settling area is not provided, high effluent suspended solids will prevail.

Completely Mixed Systems

This classification is intended to contain all those activated sludge plants which employ aeration units wherein the influent sewage is quickly and intimately mixed

into the entire aeration tank. The best known of this type is the so-called "package" type plant using a vertical draught tube. These plants had been accepted due to their structural convenience, for design flows less than 1 M.G.D.

If the micro-organisms are to operate at maximum efficiency at all times it was felt that they should be maintained in a constant state of growth rather than the feed-starve cycles of the conventional plants. In such a state, the bacteria would always be adapted to the type and concentration of organic material in the raw waste. The aeration tank also acts as a surge tank.

Dispersal of the load throughout the aeration unit relieves shock conditions, equalizes oxygen demands, and provides for uniform operation. This feature makes the process attractive to smaller installations which experience intermittent shock loads.

These units may operate on a basis of high synthesis of sludge with scheduled wasting, or on a basis of "complete oxidation" employing endogenous respiration. The only differences are in the need for sludge wasting facilities in the former case, and greater air requirements in the latter.

SUMMARY

Under optimum conditions, activated sludge can make it possible to clarify domestic sewage in approximately one half hour. It is then necessary to condition this sludge and the sewage solids it has removed, by biochemical oxidation. This does not need to take place in contact with the liquids which will become the plant effluent, and several modifications utilizing separate aeration or reaeration of this sludge have evolved. These can provide improved operations and reduced capital costs.

Since the activated sludge process and its modifications also have time-solids relationships, providing a proper F/M relation is retained, the tendency is to aerate the sludge at high concentrations to reduce aeration times and unit sizes. This will require increased oxygenation refinements.

As the sludge must still be separated from the liquids of the sewage, improvements in settling tanks or dynamic separation mechanisms must keep pace.

To date, many modifications to the process have been introduced. The Step Aeration modification with its great flexibility and Extended Aeration (Total Oxidation), with its deletion of anaerobic digesters for smaller plants, appear to be the most widely accepted of these.

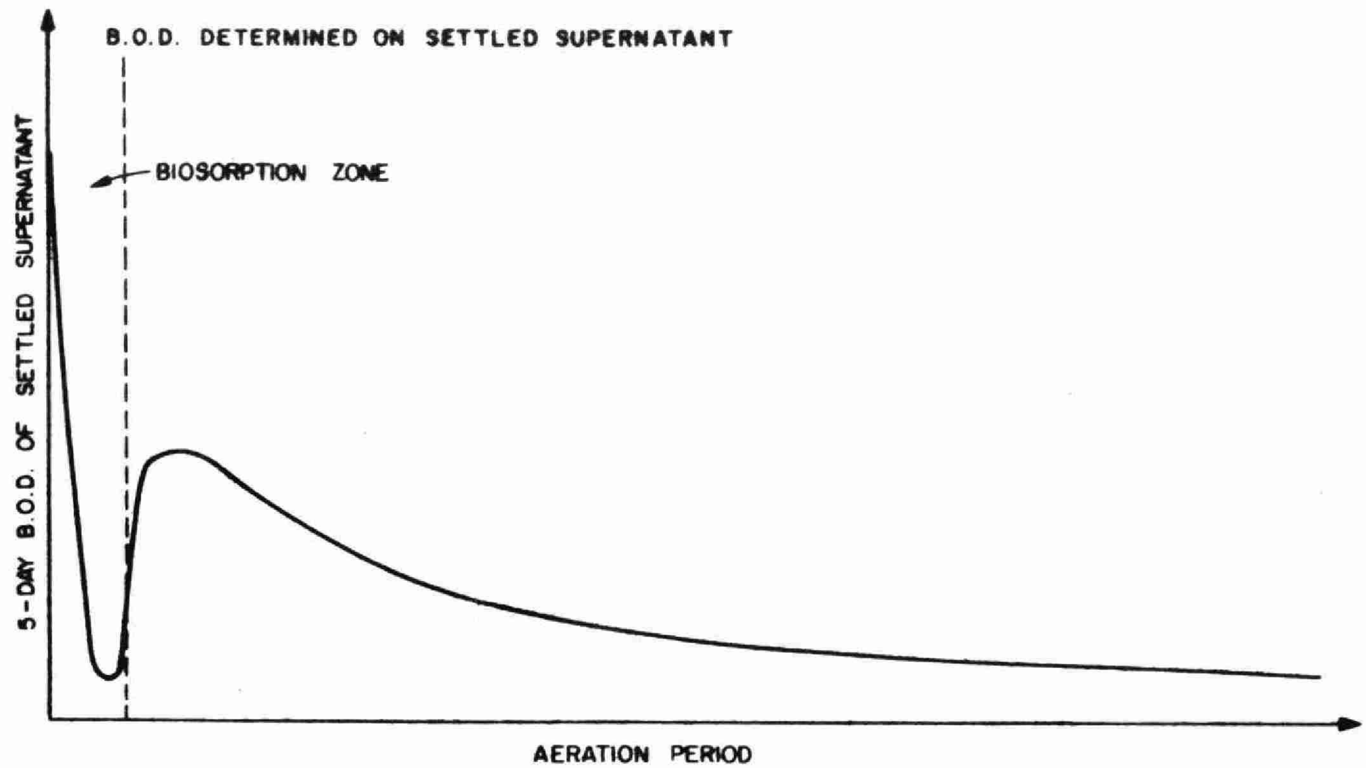


FIG. 3[A] VARIATION OF 5-DAY B.O.D. OF RAW SEWAGE-ACTIVATED SLUDGE MIXTURE WITH AERATION PERIOD.

DETERMINATION OF SLUDGE REQUIREMENTS IN THE CONVENTIONAL ACTIVATED SLUDGE PROCESS

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INTRODUCTION

The activated sludge process is utilized to convert non-settleable substances in the sewage in finely divided, colloidal and dissolved form into settleable sludge. This newly formed sludge is removed in final settling tanks and is either returned to the aeration tanks or wasted to the primary settling tanks. The raw sludge and waste sludge formed in the primary settling tanks are removed as mixed sludge to the sludge treatment facilities.

Activated sludge is a continuously changing medium in which different groups of micro-organisms primarily bacteria and protozoa in the presence of dissolved oxygen are increasing and decreasing in activity due to varying nutrients in the sewage and to varying environmental conditions. By the action of their growth these organisms convert the pollutants adsorbed on the sludge and dissolved in the sewage to carbon dioxide, sulphates, nitrates and the living protoplasm of their own bodies.

From this brief description it should be realized that the activated sludge process is one which does not readily lend itself to simple analysis and mathematical formula.

It is hoped that the following discussion of sludge requirements will assist advanced operators in the better operation of conventional activated sludge treatment plants.

AERATION TANK SLUDGE QUANTITIES

It is now recognized that the efficiency of BOD removal is related to the organic loading and the weight of solids carried in the aeration tanks.

Removals in activated sludge units average about 90 per cent at BOD loadings up to 40 pounds per 100 pounds of suspended mixed liquor solids. Small plants which do not receive 24-hour attendance should use lower loadings of only 20 to 30 pounds of BOD applied per 100 pounds of aerator suspended solids. Lower loading ratios increase the air requirements, and shift the process into the extended aeration modification operating range.

With the size of the aeration tanks fixed and the BOD loading known the amount of suspended solids required in the aeration tanks can be calculated.

For example

Sewage Flow = 1.0 MGD
5-day BOD in Raw Sewage = 200 ppm
Primary Settling Tank BOD Removal Efficiency = 35%
5-day BOD in Primary Effluent = 200 x 0.65 = 130 ppm
5-day BOD Loading to Aeration Tanks

$$= \frac{130}{10^6} \times 1{,}000{,}000 \text{ (gals.)} \times 10 \text{ lbs./gals.} = 1{,}300 \text{ lbs}$$

Assume Aeration Tank Volume = 40,000 cu. ft.

$$= 40{,}000 \text{ (cu. ft.)} \times \frac{6.25 \text{ (gals.)}}{\text{cu. ft.}}$$

$$= 250{,}000 \text{ gals.}$$

Assume Desired 5-Day BOD Loading - 20 lbs./100 lbs. of mixed liquor suspended solids

Total Pounds of Mixed Liquor Solids Required

$$= \frac{100 \text{(lbs.)}}{20 \text{(lbs.)}} \times 1{,}300 \text{ (lbs.)} - 6{,}500 \text{ lbs.}$$

Mixed Liquor Suspended Solids Concentration Required

$$= \frac{6{,}500\ (\text{lbs.}) \times 10^6}{250{,}000\ (\text{gals}) \times 10\ \frac{(\text{lbs.})}{(\text{gal.})}} = 2{,}600\ \text{ppm}$$

If the raw sewage strength increases to 250 ppm 5-Day BOD and the other factors remain the same, the mixed liquor suspended solids concentration should be increased to 3,250 ppm to maintain a steady loading ratio of 20 pounds of 5-Day BOD per 100 pounds of mixed liquor suspended solids.

It is still common practice to express the BOD loading in terms of the unit aeration capacity. Experience has demonstrated that BOD loadings of 25 to 30 pounds per day per 1000 cubic feet aerator capacity are the upper limit that can be handled in conventional plants.

In diffused air and modern mechanical aeration tanks the suspended solids concentration is usually maintained between 1500 and 3000 ppm. In older mechanical aeration plants the concentration varies between 500 and 1200 ppm.

With aerator solids held at 2000 ppm, a detention time of five hours, a settled sewage having a 5-day BOD of 100 ppm, the BOD loading on the aeration tank is 30 pounds per 1000 cubic feet and 24 pounds per 100 pounds of suspended solids. Both of these values are within the limits normally accepted as good practice.

If the sewage flow is considerably below design or the sewage strength is greater than design it becomes obvious that detention time or aerator solids must be changed to compensate.

The process should be maintained at the 5-day BOD loading to solids ratio that gives the best treatment efficiency. The optimum quantity of activated sludge carried is therefore expected to vary from plant to plant.

Although the concept of BOD loading in terms of activated sludge solids carried is an extremely important one, its application in practice must be

seasoned with a knowledge of air requirements and the ability of the final clarifiers and the return sludge system to handle the solids involved.

RETURN SECONDARY SLUDGE QUANTITIES

The inter-relationships of return sludge flow; return sludge settled volume and return sludge suspended solids with sewage flow; mixed liquor solids; mixed liquor settled volume; and the Sludge Volume Index were discussed in considerable detail during the Basic Sewage Operators Course.

The various formulae available for determining return sludge quantities are summarized in Exhibit 1.

Ideal values for the variables of the conventional activated sludge process required by the formulae were assumed to give the per cent return sludge figures shown in the summary.

There is a close correlation among the formulae although different tests are required for each.

Two formulae apply where only the suspended solids test is conducted. Two other formulae may be computed where only the 30-minute settling test is carried out. Where both of these control tests are made all five formulae may be used by the operator to establish the best return sludge rate and solids concentration for his plant.

WASTE SECONDARY SLUDGE QUANTITIES

The weight of excess secondary sludge to be wasted is dependent upon the following factors:

1. The fraction of influent suspended solids that are not bio-degradable in the process;

2. The amount of BOD_5 in the aeration tank influent converted to biological sludge;

3. The degree of auto-oxidation or endogenous respiration;

4. The concentration of mixed liquor suspended solids carried in the aeration tanks;

5. The amount of suspended solids lost in the effluent;

6. The suspended solids produced by coagulation from dissolved material;

7. The presence of adequate nutrients;

8. The temperature of the aeration tank contents;

9. The presence of toxic substances.

The first five items may be expressed directly in the following equation:

$$\Delta S = fSo + aL_r - bSa - Se$$

where ΔS = lbs. of volatile suspended solids produced per day

f = fraction of volatile suspended solids in the aeration tank influent that is not biodegradable.

So = lbs. per day of volatile suspended solids in the aeration tank influent.

a = yield coefficient, lbs. of volatile suspended solids produced per day per lb. of BOD_5 removed per day.

Lr = lbs. of BOD_5 removed per day.

b = endogenous respiration rate coefficient (1/days).

Sa = lbs. of mixed liquor volatile suspended solids under aeration.

Se = lbs. per day of volatile suspended solids lost in the plant effluent.

The sludge yield coefficient "a" for domestic sewage has an average value of about 0.35. In practice the presence of other solids in the wastes will increase this yield coefficient.

In the field, an overall sludge yield coefficient "$\bar{a}$" is used to combine the first two terms to $\bar{a}$Lr ie fSo + aLr = $\bar{a}$ Lr. The value of $\bar{a}$ is the slope of the plot of pounds of volatile suspended solids produced per day per pound of mixed liquor volatile suspended solids vs pounds of BOD_5 removed per day per pound of mixed liquor volatile suspended solids.

The sludge yied coefficient is not constant and varies with the age of the sludge, available nutrients, and presence of toxic materials. A plot of the findings of a number of investigators shows that for a sludge age of five days the volatile solids production is about 0.4 lbs. of volatile solids per pound of BOD_5 removed per day, for three days about 0.5 and for 1.5 days it is about 0.65.

Endogenous metabolism occurs in all cells in which energy is utilized for cellular maintenance. Endogenous metabolism is defined by the coefficient "b" which has the units of reciprocal time, that is a fractional decrease in cell mass per day. The coefficient "b" applies to the degradeable cellular solids. The data of Wuhrmann indicates 59 per cent active organisms at 6,000 ppm total mixed liquor suspended solids compared to substantially total activity at less than 500 ppm MLSS. Typical values of "b" for domestic sewage range between 0.020 and 0.075 per day.

The endogenous rate coefficient is temperature dependent - the higher values being obtained at higher temperatures. Also, Whurmann has indicated that the value of "b" decreases within increasing sludge concentration because of the accumulation of non-degradeable end products. In practice the value of "b" is determined from the intercept on the Y axis of the straight line fit of the plot of the actual pounds of volatile suspended solids produced per day per pound of mixed liquor volatile suspended solids vs lbs. BOD removed per lb. MLVSS.

It is apparent from the expression that the greater the concentration of volatile solids in the mixed liquor the less will be the poundage of volatiles to be wasted, provided the BOD loading remains the same.

Soluble BOD is more difficult to remove than a mixture including soluble, colloidal and suspended materials having the same oxygen demand. The process of converting dissolved material to suspended solids by biological action is a rather slow one. The amount of new sludge produced in the aeration tanks is expected to be less where the sewage being treated has a high soluble BOD concentration.

Nitrogen is one element required by micro-organisms in the activated sludge process in measurable amounts.

Systems with less than about eight ppm free ammonia as N will die because more protoplasm is degraded during endogenous respiration than is synthesized during metabolism. In this case a great build-up of polysaccharides, material resistant to biological metabolism, occurs. Because of this increase in polysaccharide material, the level of solids would increase although the system was being destroyed.

In moderate nitrogen systems containing about eight to 18 ppm free ammonia as N., there is little or no daily increase in true protoplasm and the process maintains itself. Wasting is required however to prevent the build-up of polysaccharide material.

In high nitrogen systems the high demand for synthesis is met by the formation of true protoplasm. Large quantities of protoplasm are formed at the head-end of the aeration tank. Unless sufficient air and detention time are provided to destroy the protoplasm by endogenous respiration a great increase in total sludge mass will occur, necessitating much wasting to prevent bulking and to maintain uniform conditions in the treatment process.

SLUDGE AGE

We have already indicated that the sludge yield coefficient varies with the sludge age. In other words if the sludge age is expressed in terms of BOD_5 removed and the volatile suspended solids content of the aeration tanks, other things being nearly equal, the volatile sludge accumulation should be about the same irrespective of the total mixed liquor suspended solids concentration.

Basically, sludge age is defined as the average length of time the micro-organisms are under aeration. In a system with sludge recycle such as the conventional activated sludge process:-

$$\text{Sludge Age (G)} = \frac{Sa}{\Delta S}\left(\frac{\text{LBS}}{\text{LBS/DAY}}\right)$$

Since "a", "$\bar{a}$" and "b" are not readily attainable at individual plants a simplified expression for sludge age has been used in practice as follows:

$$Sa = \frac{Va \times Ca}{Q \times Cs}$$

where

- Sa = Sludge age in days
- Va = Volume of aeration tanks in millions of gallons
- Ca = Average concentration of volatile suspended solids in the aeration tanks in ppm
- Q = Rate of sewage flow in millions of gallons
- Cs = Aeration tank influent BOD_5 in ppm

The two equations will not necessarily give the same results. Experience has shown that for recommended BOD loadings, conventional activated sludge plants should be operated with "Sa" = 1.75 to four days to keep the SVI below or near 100. Corresponding values for "G" appear to be 3.5 to nine days.

SLUDGE WASTING

To maintain equilibrium in the system as much sludge should be wasted per day as is produced. The theoretical amount of sludge to be wasted may be calculated from the previous sludge growth equations as follows:

From the previous example the following additional assumptions are made.

So (ppm) = 65
MLVSS (ppm) = 75% of 2600 = 1950
f = 0.25
a = 0.35
b = 0.055/days
Se (ppm) = 10

Since

$$fSo = \frac{0.25 \times 65 \times 1.0 \times 10^6 \text{ (gal/day)} \times 10 \text{ (lbs/gal)}}{10^6}$$

$$= 162.5 \text{ lbs/day}$$

$$aLr = \frac{0.35 \text{ (130-10)(ppm)} \times 1.0 \times 10^6 \text{(gal/day)} \times 10 \text{ (lbs/gal.)}}{10^6}$$

$$= 420 \text{ lbs/day}$$

$$bSa = \frac{0.055 \times 1950 \times 250{,}000\text{(gal)} \times 10\text{(lbs/gal)}}{\text{(days)} \times 10^6}$$

$$= 269 \text{ lbs/day}$$

$$Se = \frac{10 \times 1.0 \times 10^6 \text{ (gal/day)} \times 10 \text{ (lbs/day)}}{10^6}$$

$$= 100 \text{ lbs/day}$$

Therefore $\Delta S = 162.5 + 420 - 269 - 100$
$= 213.5$ lbs/day

[If fSo is neglected and "$\bar{a}$" = 0.485, $\bar{a}Lr$ = 582.5 and ΔS = 213.5 lbs/day]

Also Sludge Age G = $\frac{1950 \times 25{,}000\ (\text{gal}) \times 10\ (\text{lbs/gal})}{10^6 \times 213.5\ (\text{lbs/day})}$

= 10.9 days

Sludge Age Sa = $\frac{250{,}000\ (\text{gal}) \times 1950}{1 \times 10^6\ (\text{gals/day}) \times 130}$

= 3.75 days

The total weight of suspended solids to be wasted per day is $\frac{213.5}{.75}$ = 285 lbs.

Assume the return sludge contains 10,000 ppm suspended solids.

10,000 lbs are contained in 100,000 gallons

285 lbs are contained in $\frac{100{,}000}{10{,}000} \times 285$ = 2850 gal.

% Waste of raw sewage flow

= $\frac{2850\ (\text{gal}) \times 100}{1{,}000{,}000\ (\text{gal})}$ = 0.285%

Although this method is limited in its application since the assumed values of "a", "ā" and "b" may not be known at a specific plant, it may be used as a guide to proper secondary sludge wasting.

Sludge wasting at some plants is based on an arbitrary percentage of the return sludge flow. This method can be improved by using a sliding percentage scale depending on the concentration of suspended solids in the return sludge. One plant reported wasting from 3 to 5% by volume of the return sludge when the return sludge solids were in the 7,000 to 10,000 ppm range, or 0.105 to 0.175% of the sewage flow based on 35% return sludge.

L. S. Kraus (Sewage Works Journal - July, 1959) presented the following formula for sludge wasting:

$W = \frac{QcI}{10^6}$ where

W = Waste activated sludge (MGD)

Q = Raw sewage flow (MGD)

c = Suspended solids concentration of activated sludge formed (ppm)

I = Sludge Volume Index

The value of "c" to be used depends on the BOD removal in the secondary treatment units and "$\bar{a}$" the fraction of BOD removed which is synthesized to new sludge.

For Example

Sewage Flow	= 1.0 MGD
5-Day BOD in Primary Effluent	= 130 ppm (as before)
Assume 5-Day BOD in Final Effluent	= 15 ppm
5-Day BOD Removed in Aeration Tanks	=115 ppm
Assume S.V.I.	=100
Assume "$\bar{a}$"	= 0.5
Then c	= 0.5 x 115 = 57.5 ppm
% Waste Sludge of Sewage Flow	$= \frac{cI}{10^6} \times 100 = \frac{57.5 \times 100 \times 100}{10^6}$
	= 0.575%

In this formula wasting varies directly as the Sludge Volume Index. To maintain proper plant balance secondary sludge wasting to the primary tanks should be increased as the Sludge Volume Index rises.

MIXED SLUDGE QUANTITIES FROM PRIMARY SETTLING TANKS

By neglecting supernatant flow the theoretical volume of mixed sludge (raw sludge and waste activated sludge) produced in the primary settling tanks (GPD) can be determined from the sewage flow, raw sewage suspended solids concentration, primary suspended solids removal, total solids content of the mixed sludge and the specific gravity of the mixed sludge as follows:

From the previous examples:

Sewage Flow = 1.0 MGD
5-day BOD in Raw Sewage = 200 ppm
Suspended Solids in Raw Sewage = 200 ppm
Primary Settling Tank S.S. Removal Efficiency = 60%

LBS Suspended Solids Removed in Primaries Per Day

$$= \frac{200 \times 0.60 \times 1.0 \times 10^6 (\text{gal/day}) \times 10 \text{ lbs/gal}}{10^6}$$

= 1200 lbs/day

LBS Waste Activated Sludge to Primaries = 285 lbs/day from before

Total Pounds of Solids to Digester = 1200 + 285
= 1485 lbs/day

Assume Total Solids in mixed sludge = 4%
and its specific gravity is 1.0
40,000 lbs are contained in 100,000 gallons
1,485 lbs are contained in $\frac{100,000 \times 1,485}{40,000}$
= 3700 gallons/day

L. S. Kraus (Sewage Journal - July 1949) established the following mathematical relationship:

$$P = \frac{F + B}{8.4} \left(1 - \frac{0.65\ m}{d} \right)$$
$$\overline{\qquad\qquad\qquad\qquad}$$
$$p - m$$

where P = primary settling tank sludge (1000 US gal/hr)

F = Rate of fresh (raw) solids removed in primary settling tanks (lbs/hr)

B = Rate of suspended solids formed in activated sludge system (lbs/hr)

m = Digester liquor S.S. Concentration (%)

d = Waste Digested sludge S.S. Concentration (%)

p = Primary settling tank (mixed) sludge S.S. concentration (%)

This expression not only recognizes that the quantity of mixed sludge to be removed from the primary settling tanks (P) increases with raw sludge removal (F), secondary sludge wasting (B) and decreases with solids concentration (p) but also considers the effect of digester solids concentrations.

The equation derived is in the form of a hyperbola. This means that the rate of mixed sludge flow (P) approaches infinity as the solids concentration of the supernatant (m) approaches the solids concentration of the mixed sludge (p).

It has already been shown that as the Sludge Volume Index increases the volume of waste sludge and therefore the primary sludge increases.

If the system were in balance while pumping 3.5% mixed sludge to the digester and returning 0.5% (5000 ppm) supernatant and a change in SVI reduced the mixed sludge concentration to 2.5% the rate of flow of mixed sludge would increase from 8,000 to 10,000 US GPH. If the supernatant were of poor quality having two per cent solids the mixed sludge flow would increase from 14,000 to 30,000 US GPH.

This demonstrates both the adverse effect of returning poor-quality supernatant to the primary settling tanks and sudden changes in mixed sludge solids concentration.

If the flow of mixed sludge to the digester is not increased accordingly by the operator the sludge level in the primary settling tanks will increase and the sludge will ultimately leave the tanks with the primary effluent. This condition will require an increase in the waste activated sludge, which will further add to the load on the primary settling tanks. Thus, there is established a vicious cycle which cannot be remedied without losing activated sludge in the final effluent, wasting activated sludge to a lagoon or tank, and/or discontinuing the return of digester supernatant to the primary settling tanks.

EXHIBIT I

SUMMARY OF FORMULAE AVAILABLE FOR DETERMINING RETURN SLUDGE QUANTITIES

FORMULA	DEFINITION OF TERMS	ASSUMED VALUES	% RETURN SLUDGE	TESTS REQUIRED
$\frac{100B}{A + B} = M$	A = SEWAGE FLOW B = RETURN SLUDGE FLOW M = % SETTLED VOLUME OF MIXED LIQUOR	1.0 MGD ? 25%	33%	(A) TWO FLOW MEASUREMENTS (B) ONE 30-MINUTE SETTLING TEST FLOW AND SETTLED VOLUME
$(A + B)X = BY$	X = MIXED LIQUOR SUSPENDED SOLIDS Y = RETURN SLUDGE SUSPENDED SOLIDS A = SEWAGE FLOW B = RETURN SLUDGE FLOW	2,500 PPM 10,000 PPM 1.0 MGD ?	33%	(A) TWO FLOW MEASUREMENTS (B) TWO SUSPENDED SOLIDS TESTS FLOW AND WEIGHT OF SOLIDS
NOMOGRAPH	% SETTLED VOLUME OF MIXED LIQUOR - MIXED LIQUOR SUSPENDED SOLIDS - SLUDGE VOLUME INDEX -	25% 2,500 PPM 100	33%	(A) ONE 30-MINUTE SETTLING TEST (B) ONE SUSPENDED SOLIDS TEST SETTLED VOLUME AND WEIGHT OF SOLIDS
% RETURN SLUDGE $= 100 \frac{M}{R-M}$	R = % SETTLED VOLUME OF RETURN SLUDGE M = % SETTLED VOLUME OF MIXED LIQUOR	95% 25%	36%	(A) TWO 30-MINUTE SETTLING TESTS SETTLED VOLUME
% RETURN SLUDGE $100 \frac{(X-CP)}{Y-X}$	X = MIXED LIQUOR SUSPENDED SOLIDS CP = PRIMARY EFFLUENT SUSPENDED SOLIDS Y = RETURN SLUDGE SUSPENDED SOLIDS	2,500 PPM 80 PPM 10,000 PPM	32%	(A) THREE SUSPENDED SOLIDS TESTS WEIGHT OF SOLIDS

MATHEMATICAL PROBLEMS IN SEWAGE TREATMENT AND DISPOSAL

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In this lecture it will be intended to review the basic principles of algebra and geometry and study their application in the sewage treatment process. In addition, typical mathematical problems and their solutions will be presented to recapitulate some of the rules and procedures demonstrated in the previous mathematics course.

The student should, therefore, review the "Basic Mathematics" course before attempting the study of this intermediate course. A thorough familiarization of the mathematical rules and terminology is essential so that he can proceed at an expected rate of progress.

Algebra

Algebra is a branch of mathematics that deals with the relation of quantities or properties of numbers by means of general symbols. Furthermore, manipulations of these symbols are executed according to defined laws to express the mathematical statement in a desired form.

Clarification of these statements is, undoubtedly, in order. In algebra letters are used to represent an unknown quantity. For example, there is a certain number of people in a room. We let a letter say "x" represent this unknown quantity.

In some algebraic expressions there may be several unknown quantities constituting an equation. An equation is simply a statement of equality of two quantities.

<u>Example:</u>

The following algebraic expressions are equations:

$$4x + 1 = 17$$
$$9x + y = 3a$$
$$2bc + 8 = 14s$$

In most practical cases we will have one unknown quantity for each equation; however, we may have to rearrange the numbers in the equation to isolate the letter whose value is to be computed.

<u>Example:</u> Solve for x

1. $x + 3 = 9$
   $x = 6$

2. $2x + 8 = 18$
   $2x = 18 - 8 = 10$
   $x = \frac{10}{2} = 5$

The solutions to the above problems are fairly obvious. We are often faced with more complicated forms of algebraic equations for which we seek solutions. To correctly isolate a particular letter in an equation the following principles have been established to govern the rearrangement in an equation:

The values of the unknown quantities in an equation are not altered if:

(a) Equal numbers are added to or subtracted from <u>both</u> sides of the equation.

$$a + 7 = 11$$
subtract seven from both sides
$$a + 7 - 7 = 11 - 7$$
$$a = 4$$

(b) Both sides of the equation are multiplied or divided by equal numbers.

$$3a = 9$$

divide both sides by 3

$$\frac{3a}{3} = \frac{9}{3}$$

$$a = 3$$

(c) Equal powers or roots are applied to both sides of the equation.

$$x^2 = 9$$

take the square root of both sides

$$x = 3$$

A knowledge of the fundamentals of algebra is a definite asset when formulae are applied to the solutions of mathematical problems.

A mathematical formula is a general rule or principle which is expressed in algebraic symbols in the form of an equation. Formulae are often developed to simplify mathematical computations and to assist the operator in interpreting the results of the various sewage treatment plant control tests. Several formulae will be introduced in this course so a comprehension of their algebraic concepts is essential if the formulae are to be applied to specific cases.

Example:

The general formula for the sludge volume index of the activated sludge process is:

$$\text{Sludge volume index} = \frac{\%\ \text{settleable solids}}{\text{ppm suspended solids}} \times 10{,}000$$

or it can be expressed as

$$\text{S.V.I.} = \frac{a}{b} \times 10{,}000$$

where; a = % settleable solids (of the 30-minute settling test)

b = ppm of suspended solids (in the aeration tank)

We now apply this general formula to a specific condition.

(a) What is the S.V.I. if;

$$a = 25\% \text{ and } b = 3000 \text{ ppm}$$

$$S.V.I. = \frac{a}{b} \times 10{,}000 = \frac{25}{3000} \times 10{,}000$$

$$S.V.I. = 83$$

The S.V.I. is usually recommended to be in the range of 80 - 100. Therefore, the operator can assume that the sludge is in a favourable settling range.

(b) What should the MLSS content be if the 30-minute settling test is 28%.

$$\text{Assume } S.V.I. = 100$$

$$a = 28\%, \quad b = ?$$

$$S.V.I. = \frac{a}{b} \times 10{,}000$$

Rearranging the equation to isolate "b", we obtain (by rule B)

$$b = \frac{a}{(SVI)} \times 10{,}000$$

Substituting, we get

$$b = \frac{28}{100} \times 10{,}000$$

$$b = 2800 \text{ ppm}$$

(c) $4x + y = 16z + 12$, solve for x if $y = 8$, and $z = 1$

First we isolate the x

(i) Subtract y from both sides of the equation (rule A)

$$4x + y - y = 16z + 12 - y$$

$$4x = 16z + 12 - y$$

(ii) Divide the equation by 4 and cancel (rule B).

$$\frac{4x}{4} = \frac{16z + 12 - y}{4}$$

$$x = 4z + 3 - \frac{y}{4}$$

(iii) We have isolated the unknown and now we substitute.

$$x = 4 \times 1 + 3 - \frac{8}{4} = 4 + 3 - 2$$

$$x = 5$$

The latter example was more difficult than the first two; however, it illustrates the guidance of the rules presented earlier in isolating the unknown factor and then simply substituting. You may require further practice in these exercises to gain skill in rapid manipulation to isolate the unknown quantity.

Geometry

Plane geometry is a branch of mathematics that deals with the measurement and relationship of points, lines, angles and surfaces. Our interest lies in determining the area of surfaces, the volume of solids and the capacity of the various treatment units. A table listing the formulae of areas and volumes of popular geometric shapes is appended to these lecture notes for your reference.

Example:

(1) How many square feet are contained in a plot of land 400 ft. x 625 ft?

Area = 400 ft. x 625 ft. = 250,000 $ft.^2$

(2) What is the area of a circle with a radius of 10 feet?

$A = \pi r^2$

$\pi = 3.1416$ (for most practical calculations at a sewage treatment plant $\pi = 3.14$ will suffice in accuracy)

$r = 10$ ft.

$A = 3.14 \times 10^2$ ft.^{2}
$A = 3.14 \times 100$ ft.^{2}
$A = 314$ ft.^{2}

Note the importance of carrying the unit (ft^2, etc.) in our calculations.

(3) Compute the volume of a cylinder having the following dimensions;
L = 5 in., d = 4 in.

$$\begin{aligned}
\text{Volume} &= \text{Area of circular top x height} \\
&= \frac{\pi d^2}{4} \times L \\
&= \frac{3.14 \times 4^2 \times 5 \text{ in.}^2}{4} = \frac{3.14 \times 16 \times 5 \text{ in.}^2}{4} \\
&= 62.80 \text{ in.}^2
\end{aligned}$$

An important function of the sewage treatment plant operator is to keep his plant effectively tuned to operate at peak efficiency. There are various laboratory control tests which can be performed to guide the operator in this respect. However, most tests require the use of mathematics to enable the operator to interpret the results of the tests.

In addition, he should keep a check on the working load of the operational equipment at the plant. The design capacity of each piece of equipment should be recorded and tests should be routinely conducted to determine if the equipment has lost any of its efficiency. This will allow the accounting staff to consider the budgetary impact for replacement of equipment or plant expansion as far in advance as possible.

Some typical examples illustrating the need of mathematics in carrying out these duties are presented. A complete comprehension of these techniques will guide the student in conducting similar tests at his plant.

Rate of Flow

The rate of flow is the volume of water flowing past a given point in a unit of time. It is often applied when determining the flow of water in a stream or expressing pump or plant capacities.

Rate of flow = wetted area of channel x velocity of flow

$Q = v \times A$ (formula for flow in a channel)

or Q = Volume/time = V/t (formula for rating pump capacities etc.)

Example: Stream flow

(1) What is the rate of flow of water in a stream that is 3 feet deep and 5 feet wide if the velocity of the water is 2 feet per second?

$$Q = v \times A$$

where: Q = rate of discharge (ft.^{3}/sec.)
v = velocity (ft./sec.)
A = area (ft.^{2})

$$Q = 2\ \frac{\text{ft.}}{\text{sec.}} \times (3 \times 5)\ \text{ft.}^2$$

$$Q = 30\ \text{ft.}^3/\text{sec.} = 30\ \text{cfs}$$

What is a simple method of approximating the velocity of the water in a small stream?

Example: Determining the capacity of a pump

(2) What is the capacity of a pump in Imperial gallons per minute that can fill a rectangular tank 30 feet long by 10 feet wide by 6 feet deep in 1 hour and 30 minutes?

Q = Volume/time

Volume of the tank = 30 ft.x 10 ft.x 6 ft.
= 1800 ft.^{3}

Referring to the <u>conversion</u> <u>table</u> appended to the Basic Mathematics note that we multiply cubic feet by 6.23 to obtain Imperial gallons.

$$1800 \text{ ft.}^3 \times 6.23 = 11{,}214 \text{ Imperial gallons}$$

$$\begin{aligned} \text{time} &= 1 \text{ hour} + 30 \text{ minutes} \\ &= 60 \text{ minutes} + 30 \text{ minutes} = 90 \text{ minutes} \end{aligned}$$

$$Q = \frac{V}{t} = \frac{11{,}214 \text{ I.G.}}{90 \text{ min.}} = 124.6 \text{ I.G./min.}$$

(3) Sewage is entering a plant at the rate of 10 MIGD. What is the velocity in cubic feet per second through a grit channel which is 3 feet wide and 4 feet deep?

$$Q = v \times A$$

v is the unknown quantity therefore it is isolated

$$v = \frac{Q}{A}$$

To obtain the flow in cubic feet per second multiply MIGD by 1.85.

$$\begin{aligned} Q &= 10 \text{ MIGD} = 10 \times 1.85 \text{ cfs} = 18.5 \text{ cfs} \\ A &= 3 \text{ ft.} \times 4 \text{ ft.} = 12 \text{ ft.}^2 \\ v &= \frac{18.5 \text{ ft.}^3\text{/sec.}}{12 \text{ ft.}^2} = 1.54 \text{ ft./sec.} \end{aligned}$$

(4) Chlorination problems

A gas chlorinator is set to feed chlorine at 200 lb./24 hr. at a flow of 2 MIGD. The chlorine residual is 0.3 ppm. Find the chlorine demand.

Chlorine demand (ppm) = Chlorine applied (ppm) - Chlorine residual (ppm)

1 Imperial gallon of sewage = 10 lb.

$$\begin{aligned} \text{Chlorine applied} &= \frac{200 \text{ lb.}}{20{,}000{,}000 \text{ lb}} = \frac{10 \text{ lb.}}{1{,}000{,}000 \text{ lb.}} \\ &= \frac{10 \text{ lb.}}{1{,}000{,}000 \text{ lb.}} = 10 \text{ ppm} \\ \text{Chlorine demand} &= 10 \text{ ppm} - 0.3 \text{ ppm} = 9.7 \text{ ppm} \end{aligned}$$

(5) What would the chlorine feed rate have to be in the preceding example so that the chlorine residual would be 0.5 ppm?

chlorine applied = chlorine demand + chlorine residual

$$= 9.7 \text{ ppm} + 0.5 \text{ ppm}$$
$$= 10.2 \text{ ppm}$$

$$\text{chlorinator setting} = \frac{10.2 \text{ lb.}}{1{,}000{,}000 \text{ lb.}} \times \frac{20{,}000{,}000 \text{ lb.}}{24 \text{ hr.}}$$

$$= 204 \text{ lb.}/24 \text{ hr.}$$

(6) Primary Treatment

The primary settling tank at a plant serving 5,000 persons is 40 ft. long, 18 ft. wide and has a depth of 8 ft. 3 in. The daily average dry weather flow is 500,000 U.S. gallons. The length of the overflow weir is 50 ft.

Determine

A. The wetted surface area of the tank
B. The volume of the tank in U.S. gallons
C. The detention time in hours
D. The surface settling rate
E. The weir overflow rate

A. Area of tank

40 ft. x 18 ft. = 720 sq. ft.

B. Volume of tank

Volume = length x width x depth
Volume = 40 ft. x 18 ft. x 8.25 ft. = 5,940 cu.ft.

$$5{,}940 \text{ cu. ft.} \times 7.5 \frac{\text{gal.}}{\text{cu.ft.}} = 44{,}500 \text{ U.S. gal.}$$

C. Detention time

$$\text{Detention time (hr.)} = \frac{\text{Volume of tank in gal.}}{\text{flow in gal./day}} \times \frac{24 \text{ hr.}}{\text{day}}$$

$$= \frac{44{,}500 \text{ gal.} \times 24 \text{ hr./day}}{500{,}000 \text{ gal./day}}$$

$$= 2.1 \text{ hr.} = 2 \text{ hr.} + 0.1 \times 60 \text{ min.}$$
$$= 2 \text{ hr. } 6 \text{ min.}$$

D. Surface settling rate

This rate is generally expressed in gallons per day per square foot.

$$\text{Surface settling rate} = \frac{\text{Flow (gal./day)}}{\text{Surface Area (sq.ft.)}}$$

$$= \frac{500{,}000 \text{ gal./day}}{40 \text{ ft.} \times 18 \text{ ft.}}$$

$$= 694 \text{ gpd/sq. ft.}$$

E. Weir overflow rate

$$\text{Weir overflow rate (gpd per linear foot)} = \frac{\text{Flow (gal./day)}}{\text{total effective weir length (ft.)}}$$

$$= \frac{500{,}000 \text{ gal./day}}{50 \text{ ft.}}$$

$$= 10{,}000 \text{ gpd/linear ft.}$$

(7) The raw sewage entering a primary settling tank contained 250 mg/l of suspended solids. The primary tank effluent contained 110 mg/l of suspended solids. The average daily flow was 0.5 MIGD.

A. Calculate the pounds of solids removed by settling in one day.

B. Calculate the percent reduction of suspended solids by the primary treatment process.

C. Determine the total volume of the sludge in gallons if the moisture content was 95.5% (by weight).

A. Concentration of solids removed
$= 250 \text{ mg/l} - 110 \text{ mg/l} = 140 \text{ mg/l}$
Weight of sewage treated in one day
$= 0.5 \text{ million gal.} \times \frac{10 \text{ lb.}}{\text{gal.}} = 5{,}000{,}000 \text{ lb.}$
Solids removed $= \frac{140 \text{ lb.}}{1{,}000{,}000 \text{ lb.}} \times 5{,}000{,}000 \text{ lb.}$

$= 700 \text{ lb.}$

B. Percent efficiency =

$$\frac{(\text{solids in influent (ppm)} - \text{solids in effluent (ppm)} \times 100)}{\text{solids in influent (ppm)}}$$

$$= \frac{(250 \text{ ppm} - 110 \text{ ppm})}{250 \text{ ppm}} \times 100 = 56.0\%$$

C. Volume of sludge

The sludge is composed of 95.5% moisture; therefore, it must contain (100% - 95.5%) 4.5% solids.

4.5% of the sludge contents removed in one day weigh 700 lb. We wish to determine the weight (x) of 100%.

By ratio and proportion:

$$\frac{4.5}{100} = \frac{700}{x} \text{ lb.}$$

$$x = \frac{700}{4.5} \times 100 = 15{,}555 \text{ lb./day}$$

1 gallon of sludge = 10 lb.

$$\frac{15{,}555 \text{ lb./day}}{10 \text{ lb./gal.}} = 1{,}555 \text{ gal. of sludge/day}$$

Conclusion

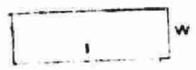
Several practical examples have been presented in this lecture to enable the operator to carry out similar calculations relating to the plant at which he is employed. A good exercise would be to calculate the actual loading rates and capacities of the various units at the plant. In addition, the percent efficiency of the treatment units could be calculated and compared to the expected or suggested ranges.

The senior session of the mathematics program will include a review of graphical representation, population equivalents of pollutional loading, averages and means and significant figures. Several examples will also be presented pertaining to the different aspects of the sewage treatment process.

Proportional Relations and Areas of Plane Figures

Rectangle:

The area, A, of a rectangle is equal to the product of its length and its width.

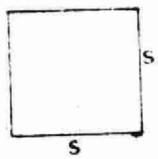


$A = lw$

Square:

The area, A, of a square is equal to the square of one of its sides.

Perimeter: The perimeter, P, of a square may be found by multiplying the length of one side by four.

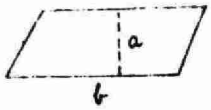


$A = s^2$

$P = 4s$

Parallelogram:

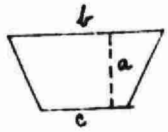
The area, A, of a parallelogram is equal to the product of its base and altitude.



$A = ba$

Trapezoid:

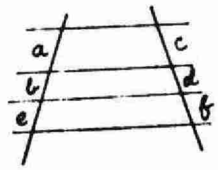
The area, A, of a trapezoid is equal to one-half the sum of its base and top multiplied by its height.



$A = \frac{1}{2}(b + c)a$

Lines:

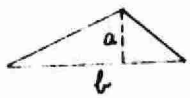
Any two lines cut by three or more parallel lines are divided into proportional segments.



$a : b = c : d$
$b : e = d : f$

Triangle:

The area, A, of a triangle is equal to one-half the product of its base and its altitude.



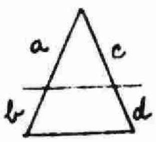
$A = \frac{1}{2} ba$

$A = \sqrt{s(s - a)(s - b)(s - c)}$
Where $s = \frac{1}{2}(a + b + c)$

The square of the hypotenuse is equal to the sum of the squares of the other two sides for any right triangle.

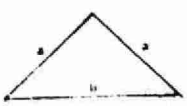
$a^2 + b^2 = c^2$

A line parallel to the base of a triangle divides the other two sides proportionally.



$a : b = c : d$
$(a + b) : b = (c + d) : d$
$(a + b) : a = (c + d) : c$

The perimeter, P, of an *isosceles triangle* may be found by doubling the length of one of its equal sides, a, and adding the length of the base.



$P = 2a + b$

Areas and Volumes of Plane and Solid Geometrical Figures

Circle:

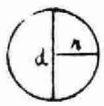
The area, A, of a circle is equal to π multiplied by the radius squared, or by one fourth the diameter squared.

$$A = \pi r^2 \text{ or } \frac{\pi d^2}{4}$$

Sphere:

The area, A, of a sphere equals the product of 4π and the radius squared.

$$A = 4\pi r^2$$



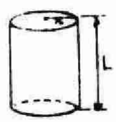
The volume, V, of a sphere is equal to $\frac{\pi}{6}$ times the diameter cubed.

$$V = \frac{\pi}{6} d^3$$

Cylinder:

The lateral area, A, of a cylinder equals 2π times the product of its radius and height, L.

$$A = 2\pi r L$$



The volume, V, of a cylinder equals π times the product of the radius squared and the height.

$$V = \pi r^2 L \text{ or } \frac{\pi d^2}{4} L$$

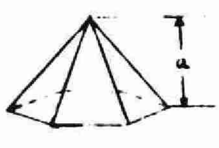
Cone:

The volume, V, of a cone equals one-third π times the radius squared times the height.

$$V = \frac{\pi}{3} r^2 a \text{ or } \frac{\pi}{12} d^2 a$$

Pyramid:

The volume, V, of a pyramid equals ⅓ times the area of the base multiplied by the height, a.



$$V = \tfrac{1}{3} \text{(area of base)} a$$

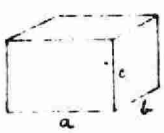
Frustum of Cone or Pyramid:

The volume, V, of the frustum of a cone or pyramid equals ⅓ height, a, times $A_1 + A_2 + \sqrt{A_1 A_2}$ where A_1 and A_2 are the area of the base and top.

$$V = \frac{a}{3} \left(A_1 + A_2 + \sqrt{A_1 A_2}\right)$$

Parallelepiped:

The volume, V, of a parallelepiped (rectangular solid) such as a cube or rectangular basin may be found by multiplying its length by its width by its height.



$$V = abc$$

AUXILIARY STANDBY POWER EQUIPMENT II (DIESEL AND GASOLINE ENGINES)

P. T. Singh
Division of Plant Operations

OPERATION AND MAINTENANCE

Regularity is the key to good operation. One should establish sound procedures and stick to them. There should be a carefully worked out routine for even simple operations as starting and stopping the engines. Determine from the engine builders' instructions and from your own experience, the various items that need regular attention during normal operation.

The auxiliary equipment should never be overlooked. The engine itself is not all of the power plant. It can only operate well if adequate care is given to the auxiliary equipment which puts it into motion and which supplies it with air, fuel, cooling water and lubricating oil.

MAINTENANCE

With the wide variety of makes and models of engines as well as the variation of climatic and operating conditions it is impossible to develop a maintenance procedure or schedule that would apply to all plants.

Hundreds of parts of the modern engines which fail or get out of adjustment do so by a gradual process. This process is speeded up by hard usage and neglect or slowed down by reasonable care and periodic maintenance. Everytime an engine operates some form of deterioration on parts take place. By controlling the rate of deterioration and by adjusting or replacing these parts before failure occurs an engine can in theory be maintained in a completely trouble-free state.

Most engine troubles can be traced to the following causes:

1. Operator's failure to follow operating and maintenance instructions;

2. Normal deterioration due to gradual wearing of parts and gradual accumulation of the by-products of combustion and the impurities in the fuel;

3. Abnormal deterioration due to part breakage.

Surprisingly a high rate of failures occur when the engine is first put into service. A complete understanding of the operating principles of each unit is necessary.

The importance of explicitly obeying the manufacturer's instruction cannot be overstressed.

Stationary plants running regularly, the operating hours, form the basis for timing maintenance operations.

For standby units, which run infrequently, but where the time element controls the amount of corrosion, sediment, etc., the customary basis is elapsed time.

Most troubles can be avoided by periodic inspection and adjustment of certain parts which experience has shown are most apt to fail.

There are two types of maintenance:

(a) Periodic inspection maintenance;

(b) A preventive maintenance programme.

Periodic Inspection Maintenance

This type of maintenance has the disadvantage

of needless disassembly of the engine when in good operating condition. It is based on inspection periods which are timed according to past performace of vital assemblies as observed by the manufacturer or the operator. These inspections require partial or complete tear down, inspection and cleaning of the engine parts with the replacement of unservicable parts. Such inspections ensure the condition of parts and assemblies.

Preventive Maintenance Programme

There is no substitute for this programme. It entails periodic inspection of specific items and an accurate record system that will indicate clearly any change in operating condition.

This system is only as good as the records which should be up to date and accurate. Records are then studied and interpreted correctly.

The main reason for preventive maintenance is to prevent an engine from running to destruction before overhaul. Most failures are preceded by definite conditions, signs or indications of approaching trouble.

An ideal preventive maintenance programme will be a guide to determine when an engine should be overhauled and to prevent serious premature engine failures.

Maintenance Pointers

A clean engine is an advertisement of good engine operation and maintenance. Too often when dirt is allowed to accumulate on the outside of the engine it finds its way into the engine interior. Dirt in one form or another is the cause of most engine failures.

Air Cleaner

This cleaner performs the very important job of keeping dirt and abrasive particles from getting into the engine. As this device removes dirt from the incoming air, it progressively becomes less effective with use in its power to clean. The air cleaner should be kept in good condition.

Fuel Supply System

Fuel combustion supplies energy to produce engine power, so the importance of a continuous supply of clean fuel cannot be over emphasized. Clean fuel improves fuel efficiency. Clean or replace fuel filter frequently. Fuel storage tanks should be checked and drained periodically to prevent any build-up of water or sediment contamination.

Lubricating System

Providing and maintaining an adequate supply of clean, high quality lubricating oil, and the proper grade in the engine crankcase is one way of ensuring long engine life and satisfactory performance. On standby engines, elapse time forms the basis to determine oil change, which includes new filter and cleaning crankcase vent.

Cooling System

Over a period of time, rust and scale accumulate in the water jackets of the cooling system. The rust and scale restrict the flow of cooling water and form a barrier to the transfer of heat from the hot metal of the block and cylinder head to the water flowing through the water jackets. The engine will tend to overheat. Rotted and partly collapsed rubber hoses, stuck thermostat, defective water pump can also restrict flow of water and cause overheating.

The cooling system should be flushed out regularly to remove accumulations of rust and scale. Cleaning compounds are available. Water should be treated with rust inhibiting compound.

Ignition System (Gasoline Engine)

In a gasoline engine, the ignition system is of considerable importance with regard to engine performance. In maintaining the ignition system to a like-new state:

1. The battery is first tested;

2. The battery cable connections inspected;

3. The ignition primary circuit is inspected and if worn or defective, restored to a like-new condition;

4. The distributor contacts are first cleaned and if necessary, adjusted or renewed;

5. The distributor or rotor and cap or terminal plate are inspected for cracks or evidence of leakage. Renew if necessary;

6. Finally, the quality of the spark at the end of each spark plug wire is determined.

Carburator

The carburator should be cleaned occasionally. Check float level, jets and fuel lines.

Fuel Injection System (Diesel Engine)

The heart of the diesel engine is the fuel injection system, because it squirts the proper amount of fuel into the combustion chamber at the right time against high internal pressure caused by compression.

The fuel system of a diesel actually performs the operation of both the distributor and carburator in the spark ignition system.

Some of the components of the fuel system are similar to those in spark ignition engines - fuel tank, filters, fuel transfer pumps and fuel lines.

Starting Battery

The battery should be checked periodically to make sure it is in a charged condition so that it can perform well when called upon to crank the engine. A battery in normal service tends to lose a little water from evaporation and from chemical action taking place inside it. This water should be replaced when necessary making sure not to overfill the cells. If cells are overfilled the acid will bubble out when the battery is being charged.

A hydrometer is used to test the specific gravity of battery. This gives a measure of the state of charge of the battery.

When the hydrometer reads:

1.265	and	above	- fully charged
1.265	-	1.235	- three-fourths charged
1.235	-	1.205	- one-half charged
1.205	-	1.170	- one-fourth charged
1.170	-	1.140	- almost run down

Caution

The surface acid in the battery electrolyte is very corrosive and will eat holes in cloth, shoes and metal. This acid can also cause painful and severe burns if it gets on your skin. It could blind you if it gets into your eyes.

The gasses that form in the tops of the battery cells during charge are very explosive. Never bring an open flame near the tops of the battery cells.

FILMS ON WATER POLLUTION

THE INVISIBLE RIVER

A history of a modern water supply system. This film deals with the task of bringing water to inland communities who have outgrown their ground water supply. The massive project is compared to a child's project on the beach; it gives a sweeping view of the wonders water makes possible.

(OWRC Production)

THE RIVER MUST LIVE

This excellent study of pollution, produced in Europe, shows how a river will die if pollution is not curbed. It outlines the causes and proposes solutions to the problem.

(Produced in Britain)

BOILERS AND HEAT EXCHANGERS
(Burners and Combustion)

J. F. Scheffers
Division of Plant Operations

In the basic sewage course we discussed the operation and maintenance of boilers. We also discussed controls and the safety devices that are associated with boilers. We have seen why good brickwork is important and what function it performs in a boiler and we have discussed maintenance and inspections. In this lecture we shall discuss burners and combustion. Different types of boilers have different types of burners; these burners can fire on oil, gas or a combination of both. Basically, however, every burner does the same thing, it combines the fuel with the air into a combustible mixture.

OIL BURNER TYPES

There are many different types of oil burners on the market, and they can be listed in three categories.

Domestic

Commercial

Industrial

Domestic: is the type used in homes, such as space heaters. (See Figure 1)

Commercial: is the type found in oil fired furnaces for the more modern homes and small establishments. (See Figure 2)

Industrial: is the type found on boilers for industrial use. (See Figure 3)

COMMERCIAL AND INDUSTRIAL OIL BURNERS

We shall concern ourselves with commercial and industrial burners. These can be split into four categories according to the way oil is applied.

(a) Atmospheric

(b) High pressure gun

(c) Rotary

(d) Air or steam atomizing

Figure 1

The space heater is a typical atmospheric burner. The air for combustion is delivered only by the draft action of the chimney and is dependent on atmospheric conditions.

Figure 2

Is the type of high pressure gun burner where air is supplied by a forced draft fan and oil is supplied under high pressure by a pump.

Figure 3

Shows a rotary type burner where oil is flung off the edge of a sort of cup rotating at high speed.

Figure 4

Shows a steam or air atomizing burner where the speed of the air or steam breaks up the oil.

In our plants we do not use the atmospheric or steam atomizing types and only in one or two places do we find a rotary burner. Therefore, we will only discuss the high pressure gun type burner.

FIG. — 1

ATMOSPHERIC POT BURNER (SPACE HEATER)

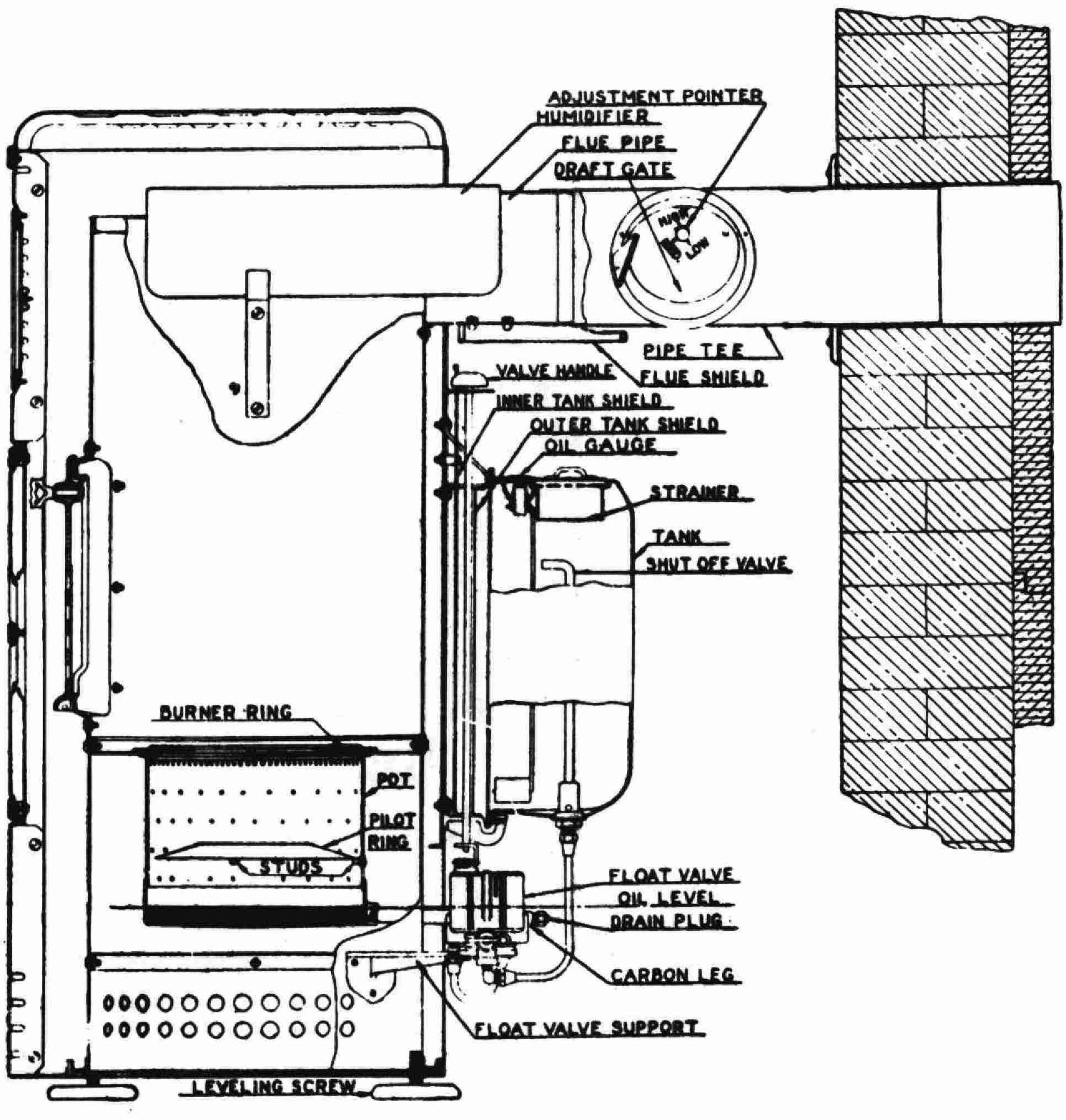


FIG. -2
HIGH PRESSURE DOMESTIC AND COMMERCIAL OIL BURNER

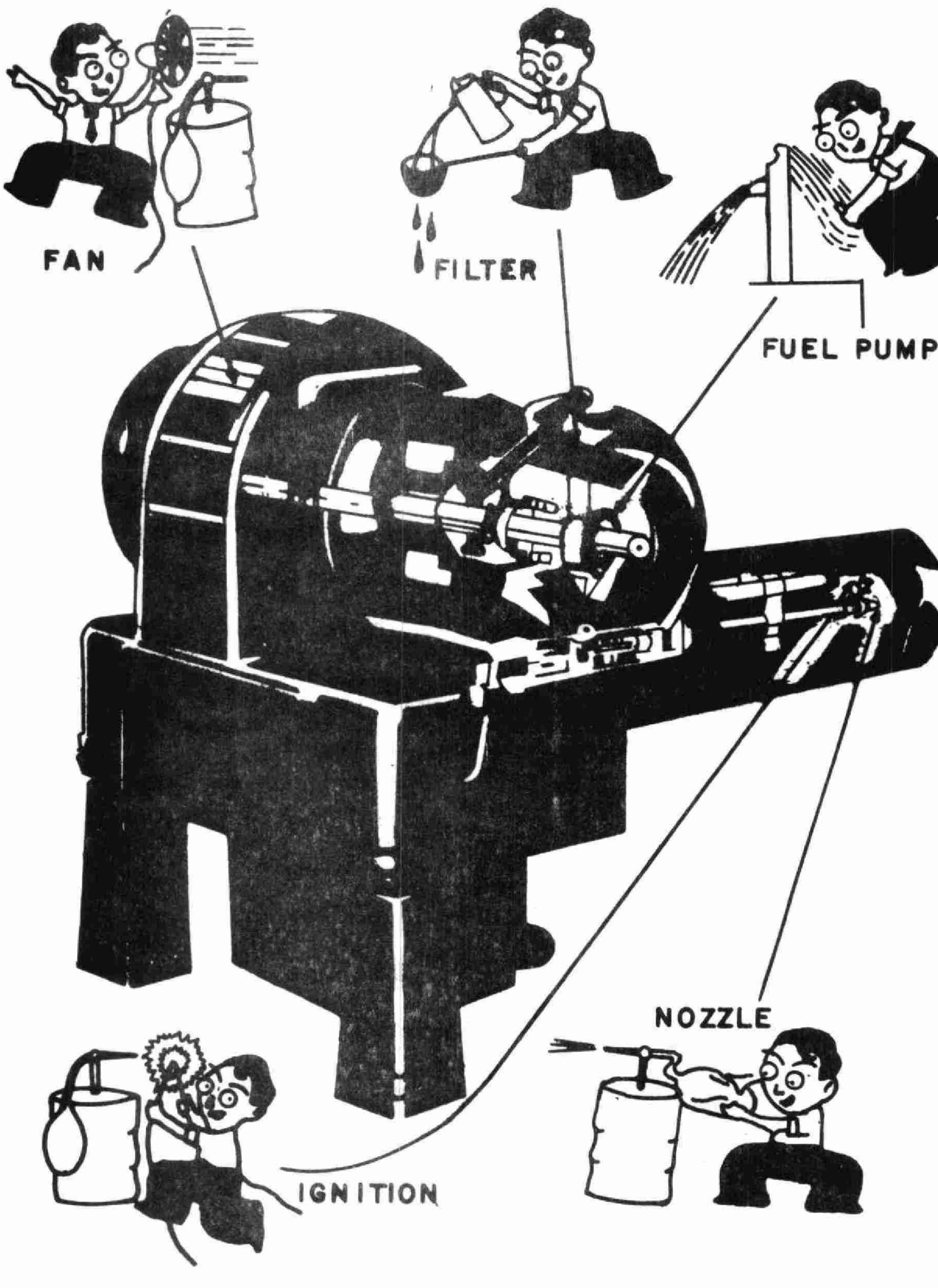


FIG. — 3

HORIZONTAL ROTARY CUP BURNER

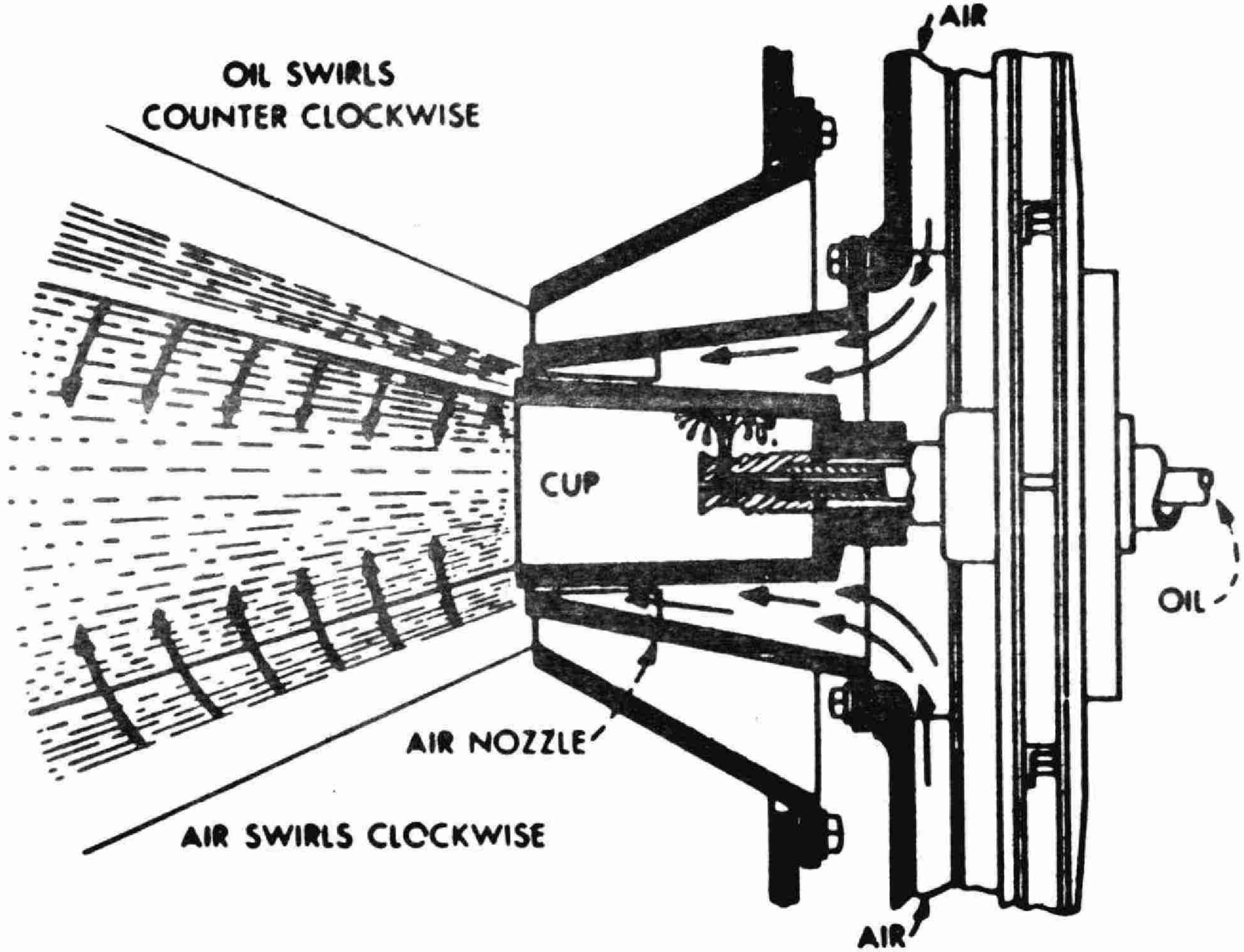
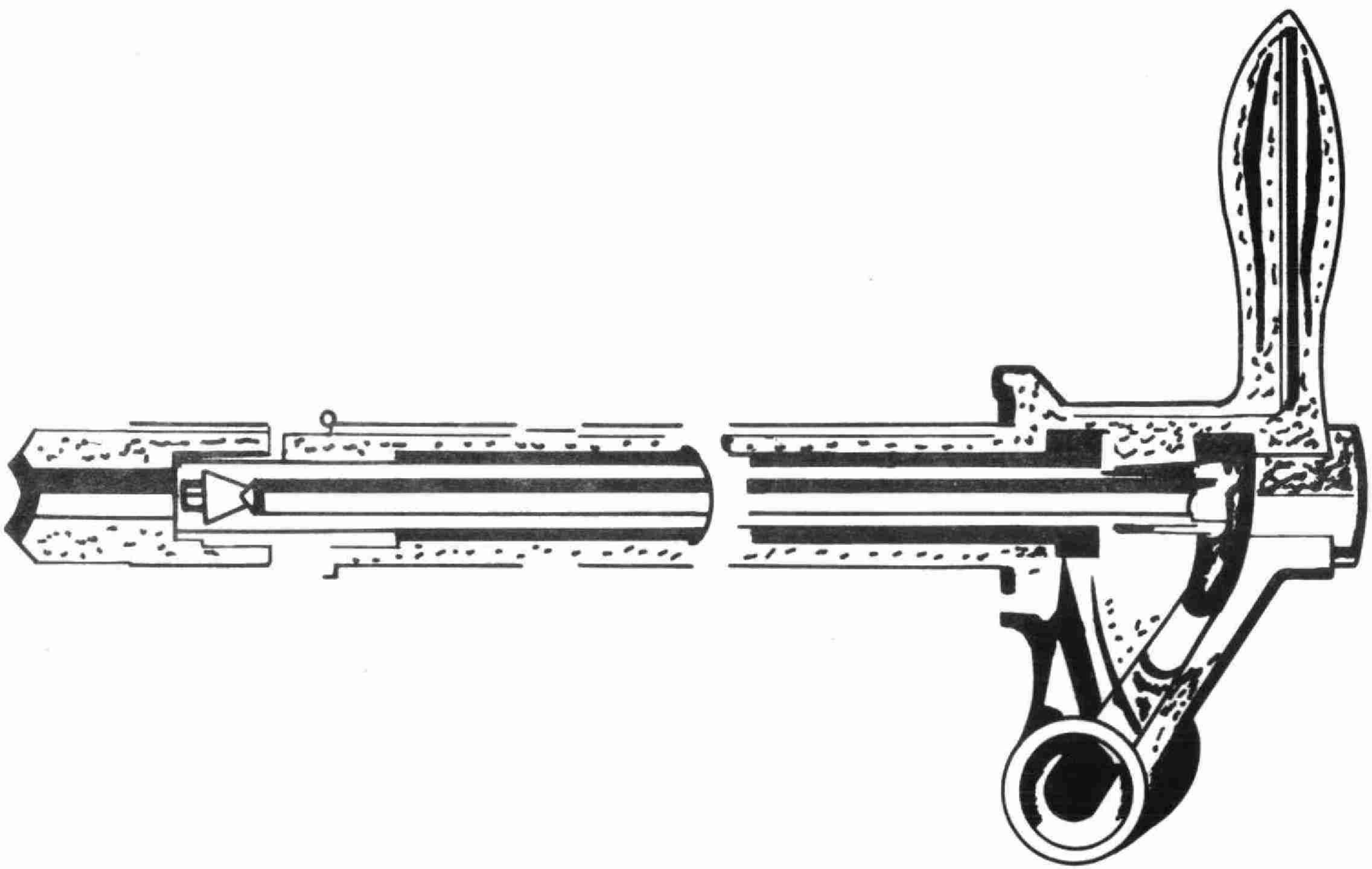


FIG — 4

STEAM — ASSISTED PRESSURE ATOMIZER



The name "oil burner" is a misnomer. It does not burn the oil, it only prepares it for burning in a combustion chamber by breaking the oil up into minute particles, almost like a vapour, and mixes this vapour with air. It further gives the oil and the air a turbulence so that the air and oil become thoroughly mixed.

ELECTRICAL PARTS OF THE BURNER (See Figure 5)

Let us look at the different parts of an oil burner. Attached to the housing and built into the unit are electrical parts which are essential to the operation of the burner. These are -

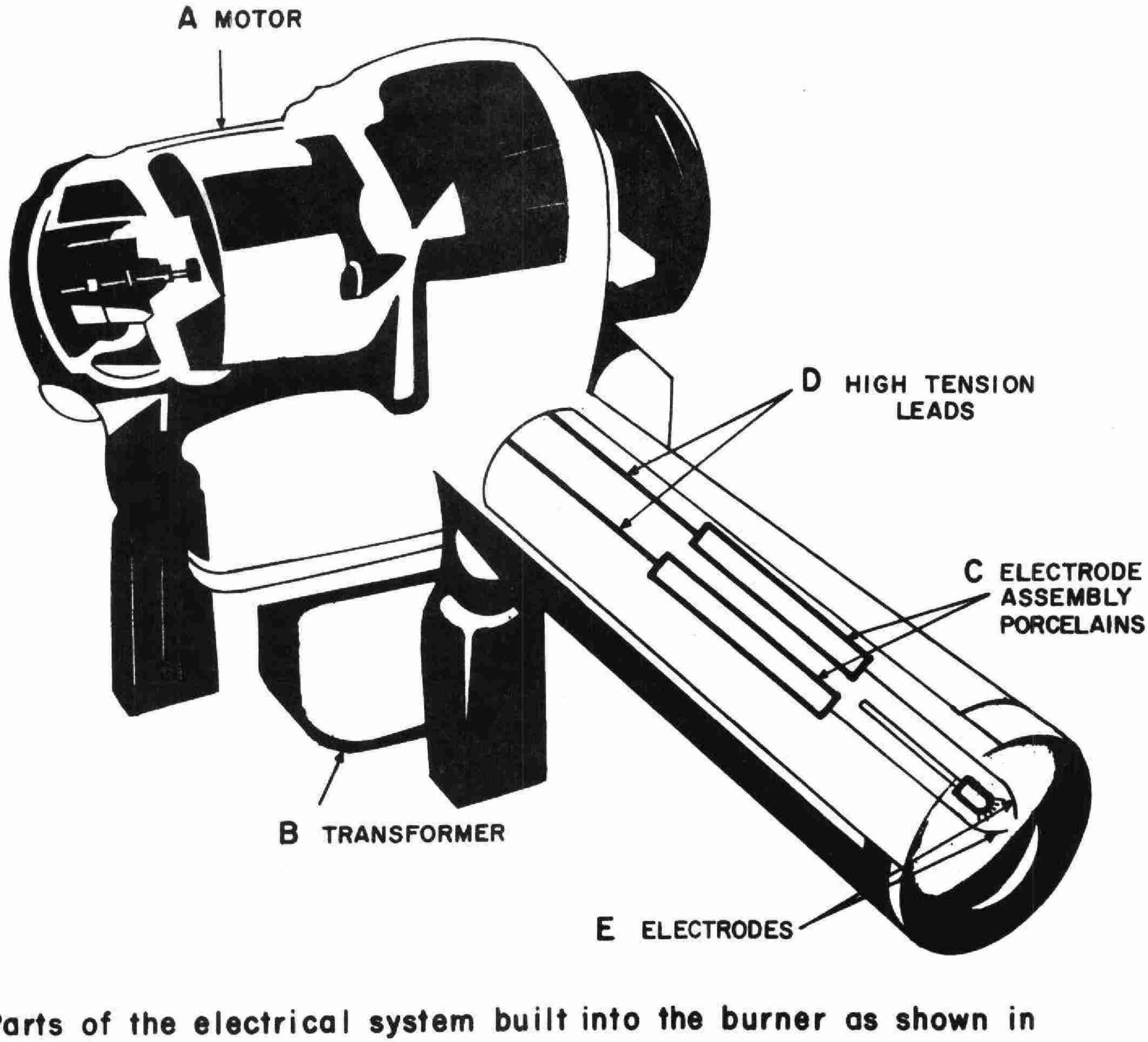
1. Motor
2. Transformer
3. High tension leads
4. Electrodes

1. The motor runs the forced draft fan and most times the oil pump. However, the latter is sometimes a separate unit with it's own motor. Let us dwell a moment on the words "forced draft". This means that the air for combustion is pushed into the boiler by a fan. This is different from "induced draft". Here a fan is placed in the stack and actually sucks the air for combustion through the whole boiler. This type of fan is not too often used because it is much larger than a "forced draft" fan and uses more power.

When a "forced draft" fan blows in air, it blows in cool air which is rather dense in structure but when this air is heated in the boiler it expands. The "induced draft" fan must handle this same amount of expanded air and, therefore, must be larger in capacity.

FIG. – 5

HIGH PRESSURE BURNER CONSTRUCTION



Parts of the electrical system built into the burner as shown in fig. 5 are : A, motor : B, transformer : C, electrode assembly : D, high tension leads: E, electrodes.

2. The ignition transformer supplies high voltage for ignition. It draws either one of 110/220 V and delivers 10,000 volts.

3. The high tension leads fulfill the same function as the ignition orspark plug wires on your car, it leads the high current to No. 4.

4. The electrodes fulfill the same function as a spark plug, namely, providing the spark for ignition.

THE ELECTRODE SETTING

This setting is very important for proper ignition. A rule of thumb for electrode setting is 1/8-inch to 3/16-inch apart; 3/8-inch to 1/2-inch ahead of the nozzle and about 3/8-inch to 1/2-inch above the centre of the nozzle. (See Figure 6)

The electrodes should not be touched by the oil spray but should be about 1/32-inch away from it. Always follow Manufacturers instructions on exact measurements for electrode setting.

MECHANICAL PARTS OF THE BURNER (See Figure 7)

1. "Forced draft" fan or sometimes "induced draft" fan.
2. Casing or blast tube
3. Turbulator
4. Fuel pump
5. Oil delivery tube or nozzle tube
6. Nozzle

The "forced draft" fan (1) delivers the air for combustion through the blast tube (2) throughout the turbulator (3) which gives the air a swirling motion to the front of the burner. The air is drawn into the back end of the burner through adjustable air shutters. The fuel pump (4) brings the oil up from the tank and brings it through the delivery tube (5) to the

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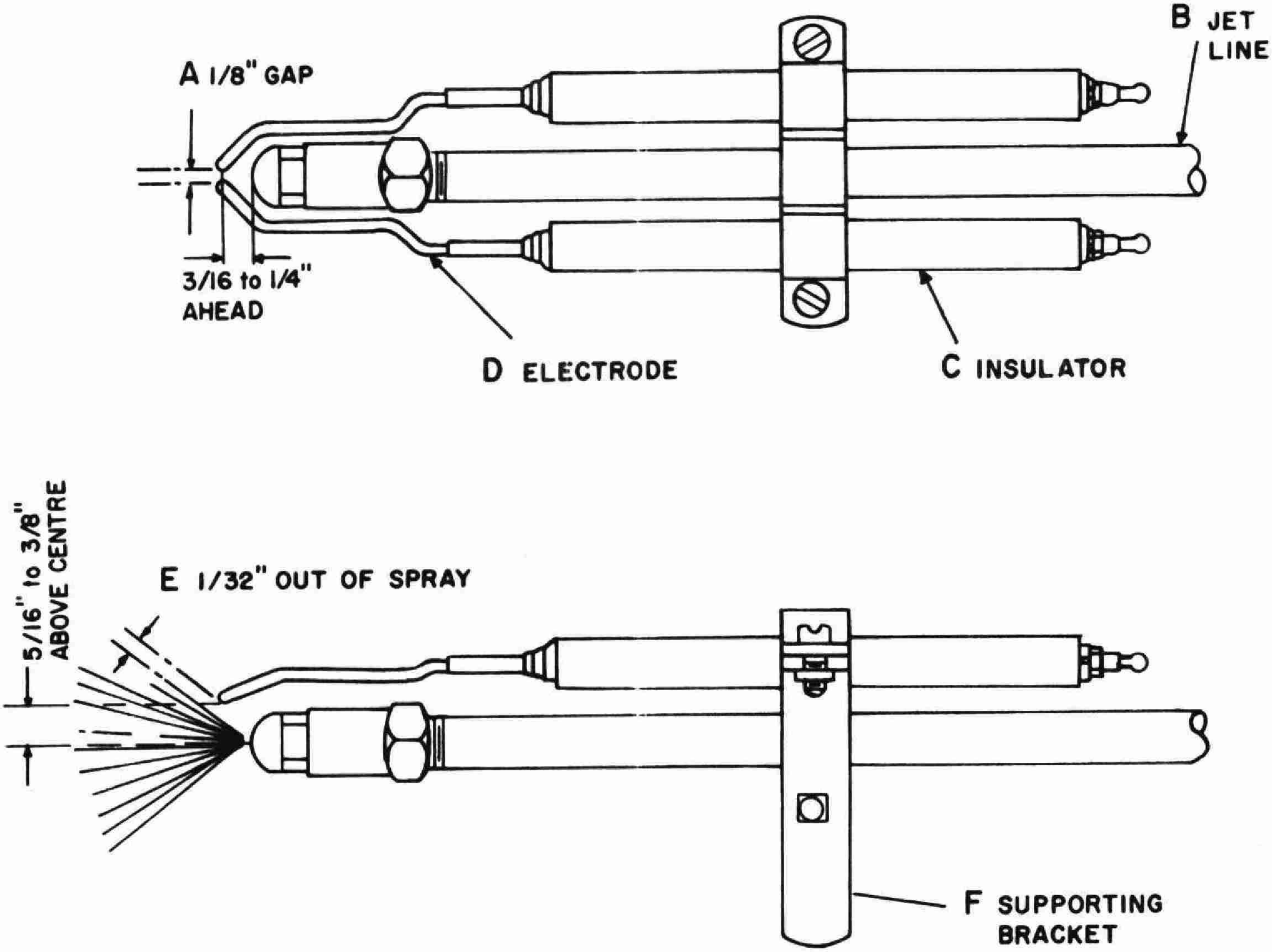
FIG.-6

ELECTRODE SETTING



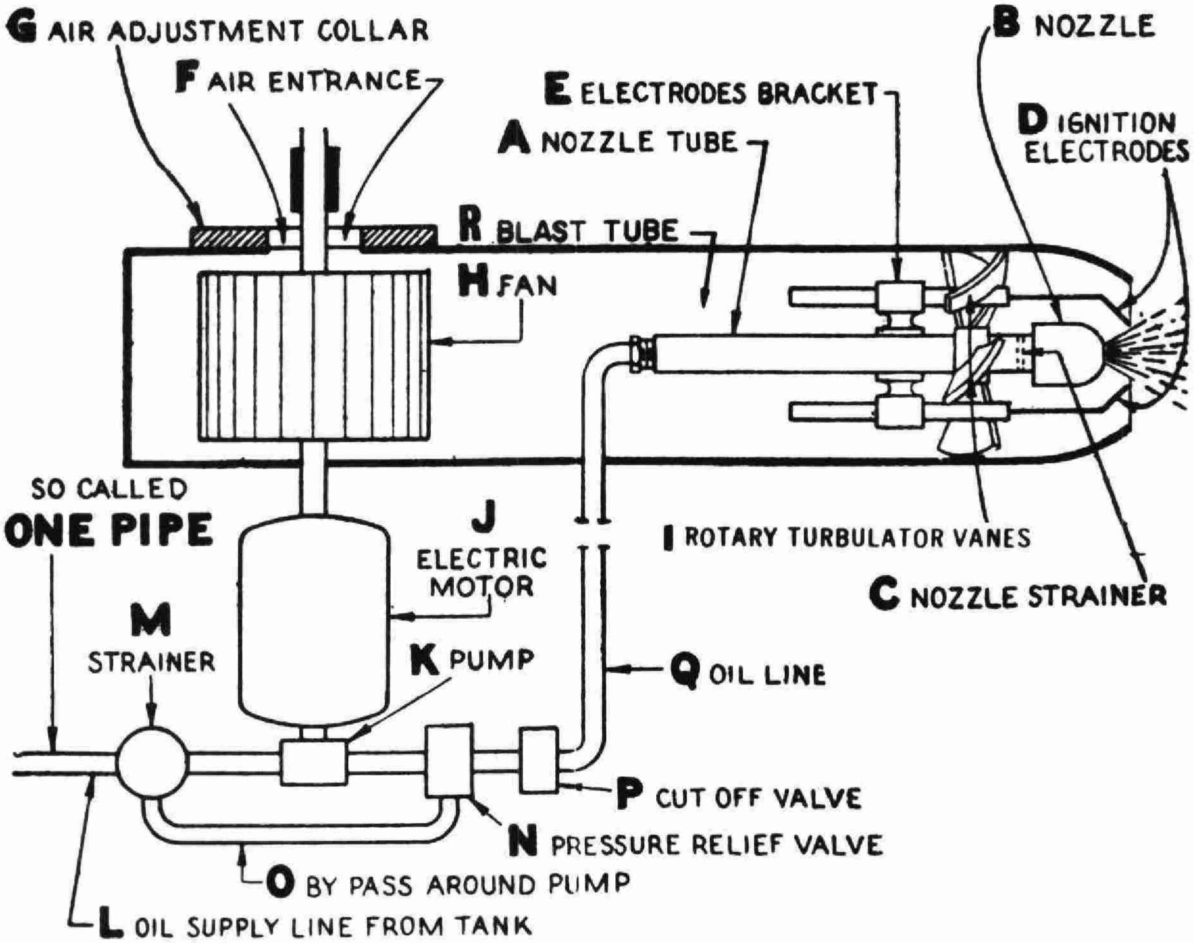
Parts of the Electrode Setting as shown in fig. 6 are :

A, 1/8" gap : B, jet line : C, insulator : D, electrode :

E, 1/32" out of spray : F, supporting bracket.

FIG. - 7

SCHEMATIC DIAGRAM OF A HIGH PRESSURE OIL BURNER



nozzle (6). The pump delivers the oil under a high pressure. This pressure is between 150 - 200 psi. An internal or external by-pass arrangement brings the nozzle pressure back to 100 psi. All nozzles are rated for this pressure.

OIL NOZZLES

Oil nozzles need a little extra discussion. A nozzle is a finely machined piece of equipment in itself and should always be treated with the utmost care. Never use any coarse materials such as wire brushes or steel wool to clean any internal or external parts. If this seems necessary, discard it and buy a new one. Even the smallest minute scratches in the oil passages can throw out the spray pattern and results in a poor fire.

To clean a nozzle soak it in a solvent such as Varsol or another of the types commercially sold. If you feel it needs further cleaning in the small passages use a wooden toothpick. Sometimes new nozzles have dried oil in the passages which throw out the pattern, clean them in the above manner.

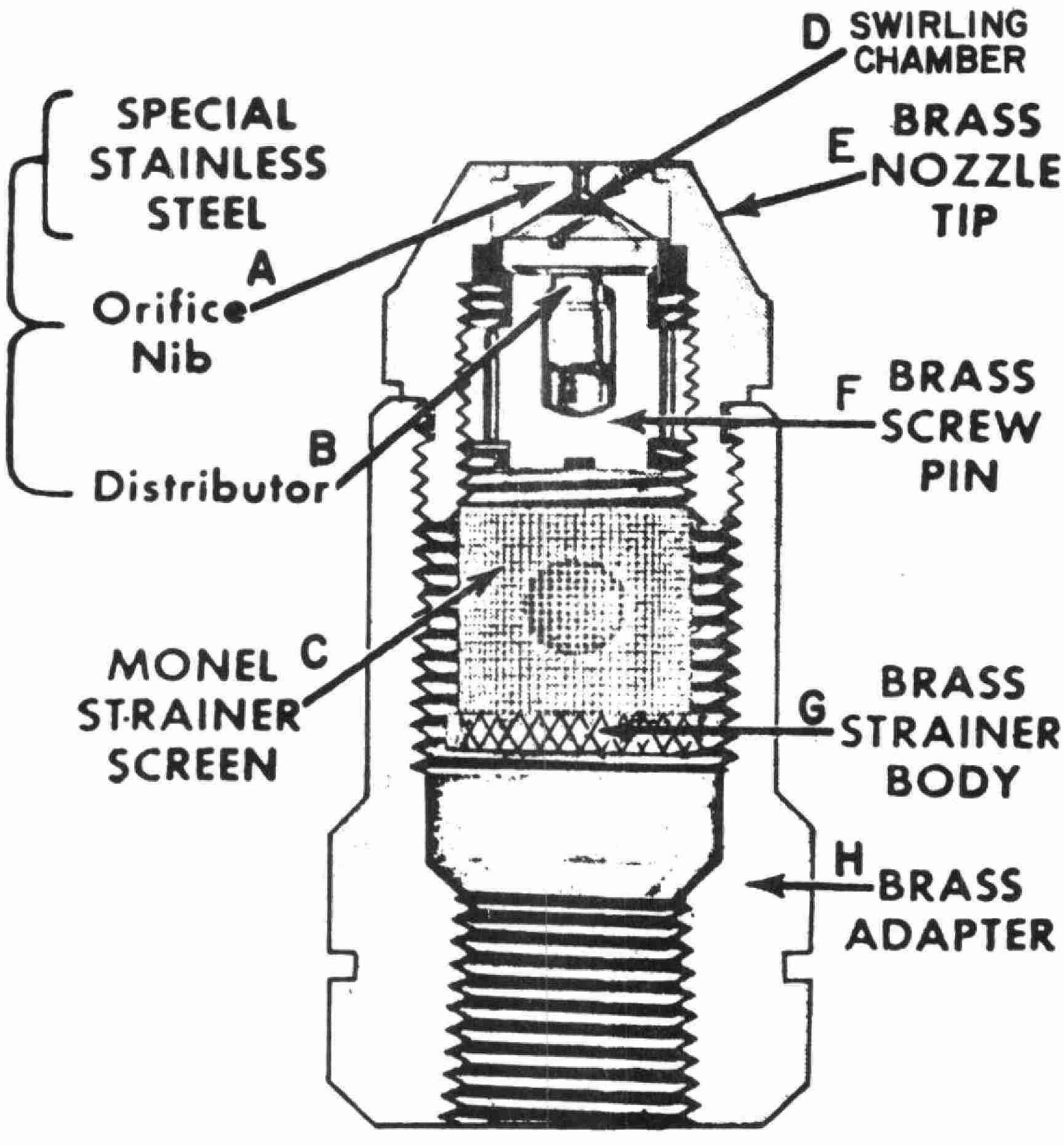
NOZZLE PARTS (See Figure 8)

The nozzle has six (6) main parts.

1. Nozzle tip
2. Distributor
3. Screw pin
4. Strainer body
5. Strainer screen
6. Adapter

FIG. - 8

HIGH - PRESSURE GUN FUEL NOZZLE



Oil delivered by the pump to the nozzle enters the filter screen and is forced through the slots in the distributor into the swirling chamber. It is the slots which give the oil a swirling motion. From here it is forced through the orifice with high velocity into the combustion chamber which it enters as a fine mist.

The chamber on the orifice denotes the spray angle of the nozzle which may be anywhere from 15 degrees to 90 degrees. The diameter of the orifice itself determines the size of the nozzle.

Just at the front of the nozzle the air is brought in contact with the oil and this air has also a swirling motion in opposite direction of the oil. The result is a thoroughly mixed mixture of air and oil which will easily ignite the moment the spark is applied.

All nozzles are rated in U.S. gallons per hour. When you see 5.00 - 30^{o} stamped on the side of the nozzle it means the nozzle is rated at 5 U.S. gallons per hour and has a spray angle of 30 degrees at 100 psi. Always use the size of the nozzle specified for your particular boiler. If you use a smaller size you will not be able to get the maximum heat out of your boiler. If you use a larger size, there is danger that sufficient air will not be available for combustion and improper firing will result. There is some flexibility in the oil pressure if just a little more heat is required. The oil pressure may be boosted to 125 psi which will give more oil but do not go below 100 psi because this results in poor mixing with the air.

TYPES OF FUEL AND THEIR HEATING VALUE

In our plants we use three (3) types of fuel:

1. #2 Furnace oil

2. Natural gas

3. Digester gas

We rate fuels by their heating value or as we often say "B.T.U." value. What is B.T.U.? It stands for British Thermal Unit. One B.T.U. is equivalent to the amount of heat required to raise the temperature of one pound of water one degree Fahrenheit in temperature. Following are some heating values for different fuels.

Furnace oil	-	18,000 B.T.U./lb or 140,000 BTU/USG
Natural gas	-	1,050 B.T.U./cu.ft.
Digester gas	-	450 - 650 B.T.U./cu.ft.
Coal	-	14,000 - 18,000 B.T.U./lb.

Many boilers use two types of fuel and combinations such as oil and digester gas. It is often difficult to adjust a combination burner such as oil/gas for the same air setting without instruments, only a happy medium can be obtained at best.

Why do we need air? All we want out of the air is only one element. Oxygen. Most of you will know that the greatest part of air is Nitrogen (79 per cent) and the smallest part is oxygen (21 per cent) by volume. This brings us back to that magic work "combustion".

What is combustion? Combustion is the rapid oxidation of fuel. The carbon and/or methane gas in the fuel combines in a rapid chemical reaction with the oxygen of the air, with the evolution of heat and often light. A very rapid combustion is called an explosion, while on the other hand rusting is also a form of combustion, because here oxygen is combining with metal. You can see there can be a big difference in the speed of combustion. Normally we understand the word combustion to being related to the burning of fuel with the

purpose of providing heat, power or both. The nitrogen in the air is actually a nuisance to us. We have to heat it up to combustion temperature and then let it escape up the stack, unused. Some of this heat is recovered as it passes over the boiler heating surfaces but we still lose the 400 degree to 500 degree temperature that is left.

Temperatures in the combustion zone vary greatly depending on the type of fuel that is used. Temperatures vary from 1,800 degrees F. to 3,000 degrees F. Gases will leave the stack at 300 degrees to 600 degrees after giving up the heat to the boiler heating surfaces. This, of course, represents a loss of some 250 degrees to 550 degrees F. There is a difference between the actual stack temperature and the actual heat loss because the air temperature when it was first drawn into the boiler was around 50 to 70 degrees F. This is called ambient temperature and this is always taken into consideration by combustion calculations.

The stack loss is one thing we can do nothing about. It represents a certain percentage of the heating value of the fuel. Due to this loss we can now see that it is impossible to obtain a 100 per cent efficiency in combustion. An efficiency of 80 per cent is considered good. Good combustion requires four (4) pre-requisites and the following word might be helpful in remembering them - MATT.

M for mixture (of air and fuel)

A for air (to obtain oxygen)

T for temperature (to obtain ignition)

T for time (for the air to mix with fuel)

If any of these four are absent combustion is not possible. When the carbon (C) in the fuel combines with oxygen (O_2) in perfect combustion another gas is created called Carbon Dioxide (CO_2). When combustion is incomplete (smokey fire) a gas called Carbon Monoxide

(CO) is created. In other words, all the carbon is not burned. We like to get away from the latter and to get a high as possible (CO_2) content in our flue gases for complete combustion. This CO_2 content in the flue gases provide us with an excellent guide for our combustion efficiency.

When we talk about percentages we always think of 100 per cent as the ideal figure. This is not so in combustion. Just look back at the composition of air, 21 per cent was the amount of oxygen in the air and this is all we are using. Granted, a volume of flue gas is 100 per cent but of this 100 per cent, 79 per cent is nitrogen and useless to us and the remaining 21 per cent CO2 is the ideal figure in combustion. If each particle of carbon would combine with the exact right amount of oxygen we would get 21 per cent CO_2. However, this is impossible to obtain.

To be sure that enough oxygen is available to consume all the fuel we have to add more air than is theoretically necessary. Some of the air will not mix with fuel and passes up the stack heated and due to mechanical difficulties it is impossible to measure the right amount of air. We are actually using from 25 per cent to 50 per cent excess air. This shows up in the flue gas analysis as O_2 and brings down the CO_2 percentage. A CO_2 content in the flue gas of 10 per cent to 12 per cent with a stack temperature of 400 to 450 degrees F. is considered good for oil. For gas this figure lies between 8 per cent to 10 per cent with a stack temperature of 300 to 350 degrees F.

GAS

For the combustion of gas the process is much the same. The only thing we do not have to do is to atomize it because it is already a gas and will mix readily with the air. The components of gas differ from oil in this respect, that oil has carbons and carbohydrates while gas is mainly methane, 93 per cent by volume for natural gas.

Digester gas is between 40 per cent and 60 per cent methane with the other part being CO_2 and small amounts of other gases.

When methane (CH_4) combines with O_2 two different gases are formed, CO_2 and H_2O carbon dioxide and water vapour. For this reason we can always expect a lower CO_2 content in the flue gas when burning gas because part of the flue gases is water vapour (H_2O). This vapour is super-heated and passes up the stack as steam. That is the reason why on a cold day you always see "white smoke" coming out the stack. This is called condensation.

So much for burners and combustion. If you have any questions I will try to answer them.

DEWATERING OF SLUDGE BY VACUUM FILTRATION

by

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GENERAL

Sewage treatment is concerned with the extraction of solids from sewage and the ultimate disposal of these solids. The dewatering of solids consists of plain sedimentation, sometimes digestion of the collected sludge in order to stabilize and render it inoffensive, drainage on sand beds or mechanical filtration and finally disposal on the land where evaporation continues the drying process.

Plain sedimentation removes 99.7 percent of the total water, digestion and sand bed dewatering or mechanical filtration remove approximately another 0.3 percent of the total water. Actually, about 0.04 percent of the total water present in the incoming sewage still remains after sand bed dewatering or mechanical filtration and this is sometimes removed by incineration.

Vacuum filters were utilized to dewater waste activated sludge at Milwaukee in 1925 and this was the first application of the filters in the sewage field in North America. However, "Drum Filters" were first invented in England in 1872 for cement slurries. The filters at Milwaukee used sulphuric acid as a coagulant, then chlorinated copperas, and finally ferric chloride.

A vacuum filter is a rotary drum under vacuum, suspended over a trough or vat which contains wet sludge ready for filtration. Appurtenances included with the filter are a vacuum pump, filtrate receiver, filter media, washing showers to clean the filter media, belt conveyors to remove the dried sludge and a hopper or storage area. Storage and pumping facilities for the coagulation

chemicals and a mixing tank for the wet sludge and these chemicals are also required.

Various types of filter media have been employed since 1925. The ideal media is one that offers no resistance to the flow of liquid, but serves as a support for the solid particles forming the cake. The following materials are those most commonly used as filter media.

1. Fabric Covered Filters

The filter cloth is made from cotton, wool, felt, dacron, saran, polyethulene and many others. The synthetic fibres have longer life, greater yield in many cases and are easier to clean. Woven wool is an excellent media while new, but clogs with fine particles and conditioning chemicals very quickly. The expected life for wool is from 250 to 800 hours with an anticipated life of 10,000 hours for dacron and saran.

2. String Filters

These filters use a fabric media but the sludge cake is removed from the drum by strings which pass around it. No air blower or scraper is required.

3. Travelling Belt Filters

These consist of a stainless steel woven wire belt which serves as the filter media.

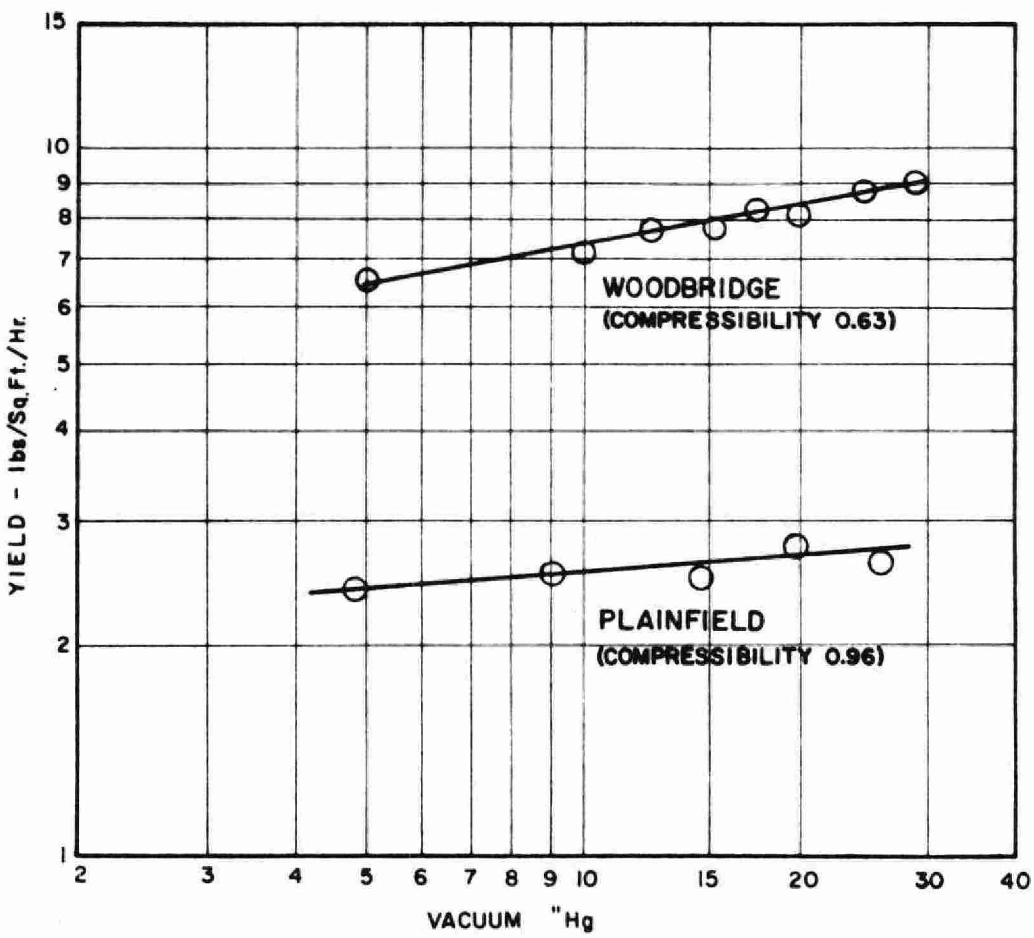
4. Coil-spring Filters

Two layers of coiled steel spring are placed in corduroy fashion around the drum. As the layers leave the drum, they are separated from each other and the filter cake is lifted off and discharged.

Raw, digested primary and/or activated sludge can be dewatered by vacuum filtration. The amount of chemical coagulants and the yeld (pounds of dry solids per square foot of filter area per hour) depends to a large extent on the type of sludge to be filtered. For instance, raw sludge filters with less chemical coagulants than does digested activated sludge and usually at a higher rate.

FIGURE 1

EFFECT OF VACUUM ON FILTRATION RATE



SLUDGE CHARACTERISTICS AFFECTING FILTERABILITY

1. Size, Shape, Density and Charge of Solid Particles.

Irregular shaped small particles tend to form a very tight mat with only a few voids for the migration of liquid. On the other hand, regular shaped larger particles tend to give a high ratio of voids. As a result, sludge consisting largely of small particles requires the greater amount of coagulant per unit weight of solids. It was found at the Hyperion plant in Los Angles that thermophilic digestion produced larger particle sizes than did mesophilic. Digestion at the higher temperature increased the filter yield three-fold and decreased the chemical coagulant demand by 50 percent.

Sludge particles have been found to be negatively charged with a charge of about the same magnitude as that associated with proteins. In order that these particles will coagulate, it is necessary to neutralize this negative charge. Ferric chloride with three positive charges is most effective in neutralizing the negative charges of the sludge particles.

2. Compressibility of Solid Particles.

Compressible sludges tend to deform with pressure and form a tighter filter cake. Thus an increase in pressure or vacuum will not result in a proportional increase in filter rate for a compressible sludge. A compressibility of 1.0 means that the filter rate remains constant with an increase in pressure. In general, the compressibility of most sludges has been found to be approximately 0.8. Thus, a large increase in pressure will result in only a small increase in yield. This is demonstrated in Figure 1.

3. Viscosity of Filtrate and Sludge.

The viscosity of the filtrate varies with temperature. At 55 degrees centigrade, for

instance, the viscosity of water is one half of that at 20 degrees centigrade. However, it is probable that it would not be practical to attempt to raise the temperature of the sludge being filtered due to the cost involved and furthermore an elevation of temperature may produce changes in the sludge.

4. Chemical Composition

The chemical composition of sludge determines to a large extent the amounts and type of coagulating chemicals required. The greater the alkalinity of the sludge, the greater is the chemical demand; and the greater the amount of volatile matter, the greater is the chemical demand. These relationships are expressed as liquid demand (alkalinity demand) and the remaining demand is termed the solids demand.

Digestion of sludge increases the liquid demand, but decreases the solids demand. In decomposing the volatile portion of the raw sludge, the bacteria convert the putrescible compounds on the one hand to methane and carbon dioxide and on the other hand to ammonia. The carbon dioxide and ammonia combine in the sludge liquor to increase the alkalinity related to the remaining solids. In terms of solids present, the alkalinity is increased by 10 to 20 times what it was in the fresh sludge.

5. Solids Concentration.

Increasing the solids concentration will increase the yield of a filter. The more concentrated the sludge is the less filtrate volume has to be removed per pound of filter cake deposited. Concentrating also lessens the liquid demand and, hence, lowers the amount of chemical coagulants required.

CHEMICAL CONDITIONING

The amount and type of chemical coagulants is determined by the physical and chemical characteristics of the sludge. Sludge solids are primarily thought of as complex colloid systems with water as the dispersion medium. The majority of these colloids remain in dispersion due to (1) electrical charges surrounding the particle and (2) the shell of water around the particle. In order to reduce the stability of these colloids one or both factors must be removed at least partially.

The stability of the electrical charge is due to (1) ionization and/or (2) absorption of solution ions on the particle surface. Both of these factors are affected by pH, ionic strength and the type of ions in the surrounding water.

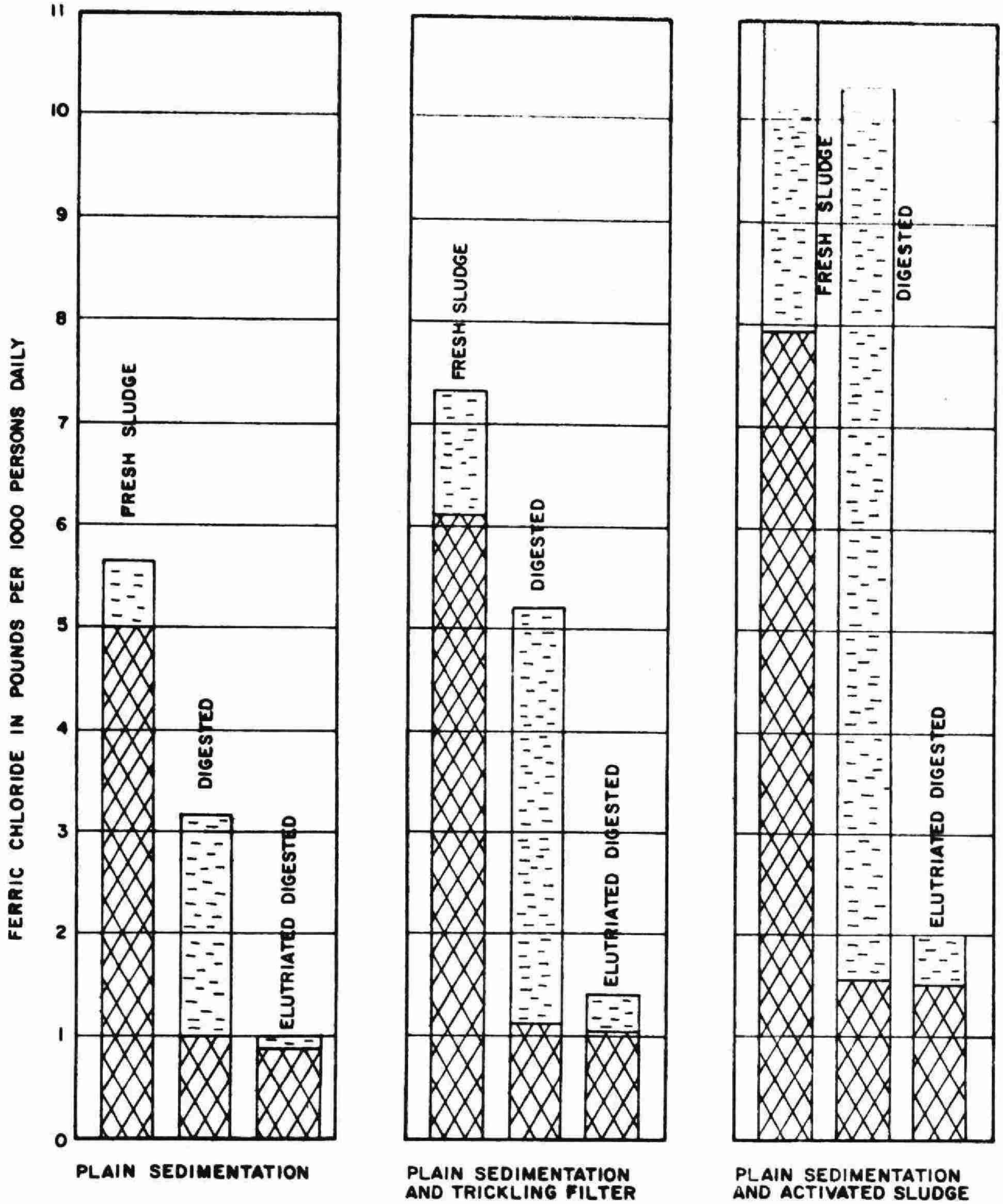
The stability of the shell of water around the particle is dependent on the structure of the molecule and the functional groups of the molecule.

Thus, any force which will neutralize the negative surface charges and/or bring about reactions with or among the functional groups can be expected to destroy the stability of the colloid systems in a sewage sludge.

The three positive charges of the ferric ion are most effective in the charge neutralization effect whereas some functional group reactions take place with $FeCl_3$ at an acid pH, others take place with lime at an alkaline pH, while a third can react with either chemical. Thus, coagulating chemicals are involved in a number of reactions with sewage sludge to make them filterable. The predominant reaction or set of reactions will vary with the type and source of sludge.

In the treatment of domestic sludges, there are three basic types of sludges. The first type, from plain sedimentation, is the easiest to handle since it is possible to obtain high concentrations both with the raw and the digested sludge. Sludges from trickling filters are more difficult to concentrate. Sludges from activated sludge treatment are usually the most difficult to concentrate due to the voluminous and spongy nature.

FIGURE 2



QUANTITY OF FERRIC CHLORIDE REQUIRED FOR VACUUM FILTRATION OF VARIOUS SLUDGES

The bargraphs in Figure 2 show the relationship between ferric chloride demand and type of sludge. This graph was presented by Genter (1) and represents the amount of ferric chloride in pounds per 1000 persons required for the three basic types of sludge. The bar graphs are based on sludges collected from separate sewage systems handling domestic sewage at a flow of 80 gallons per capita daily.

The cross-hatched portion of the bar represents the solids demand while the dashed portion indicates the liquid demand. When the percentage of alkalinity on solids is relatively low, as with most of the fresh and elutriated digested sludges, most of the chemical is for solids requirement. The digested sludges display a reduction in solids demand due to the decrease in volatile to ash ratios but havea greater liquid demand due to the increased alkalinity resulting from the end products of digestion.

The graph for the activated sludges indicate that there is only a slight reduction in chemical requirements when these sludges are digested. Sludges of this type can decrease their chemical requirements appreciably by elutriation.

ELUTRIATION

Elutriation is a solids washing process. An elutriated sludge is one that has had the alkalinity of its fouled water (from decomposition products of digestion) reduced by dilution, sedimentation and decantation in water of lower alkalinity.

Elutriation involves four essential steps to improve the biochemical quality of sludge water:

1. Dilution with water of lower alkalinity than the fouled sludge water.

2. Mixing the sludge and dilution water to produce a more dilute solution of the dissolved decomposition products present in the fouled sludge water.

3. Sedimentation of the sludge solids in the more dilute solution.

4. Decantation of as much of the weaker, relatively clear solution as possible.

Since these steps usually result in incidentally washing the sludge solids free of entrained gases the elutriated sludge gains in specific gravity and concentrates to a higher degree than the digested sludge to elutriation.

For single stage elutriation, the following formula may be employed to determine the resulting alkalinity:

$$E = \frac{D + 2W}{R + 1}$$

where D is the alkalinity in the digested or stale raw sludge water before elutriation, E is the alkalinity of the elutriated sludge, W is the alkalinity of the elutriating water and R is the ratio of volume of elutriating water used to the volume of moisture in the sludge. Example:

> Assume a digested sludge having a 4 percent solids and an initial alkalinity (D) of 3,000 ppm. The wash water alkalinity (W) is 120 ppm and the metered ratio of wash water to sludge volume is 4:1.
>
> Then $R = 4 \div 0.96 = 4.17$
>
> $$E = \frac{3{,}000 + 2 \times 120}{4.17 + 1} = 678 \text{ ppm}$$
>
> This represents about a 78 percent removal of the initial alkalinity.

OPERATING VARIABLES

A number of variables confront the operator of a vacuum filter. He can change drum speed, amount of vacuum, chemical dosages and chemical ratios (if adding both lime and ferric chloride).

Practically all of the proper techniques involved

with these variables have been developed through operating experiences at a number of plants. Certain procedures with respect to vacuum filtration often seem trivial but have been found by experience to be quite important.

1. Sequence of Addition of Chemicals.

This variable would appear quite insignificant to the laymen but it is quite important with respect to the total amount of chemicals required for coagulation. Experiments based on the Cuchner funnel test indicate that for dosages for the ferric chloride below 5.5 percent, it is important to add the ferric chloride ahead of the lime. Savings in ferric chloride and lime requirements in the order of 50 percent can be experienced by following this procedure. This is demonstrated in Figure 3.

2. Vacuum.

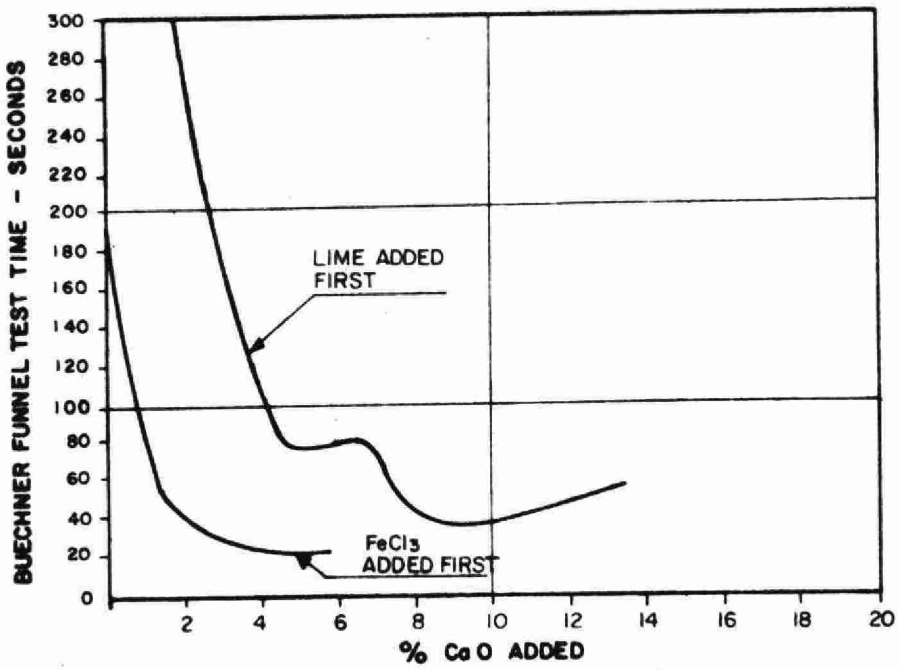
Increasing the vacuum will result in a greater removal of moisture but with respect to filter yield vacuums in excess of 10 to 15 inches of mercury are not required. This is demonstrated in Figure 1. The sludge with a compressibility of 0.96 shows a 10 percent increase in yield with a four fold increase in vacuum. The other sludge, which is less compressible (0.63), shows a 32 percent increase in yield with a four fold increase in vacuum.

3. Mixing.

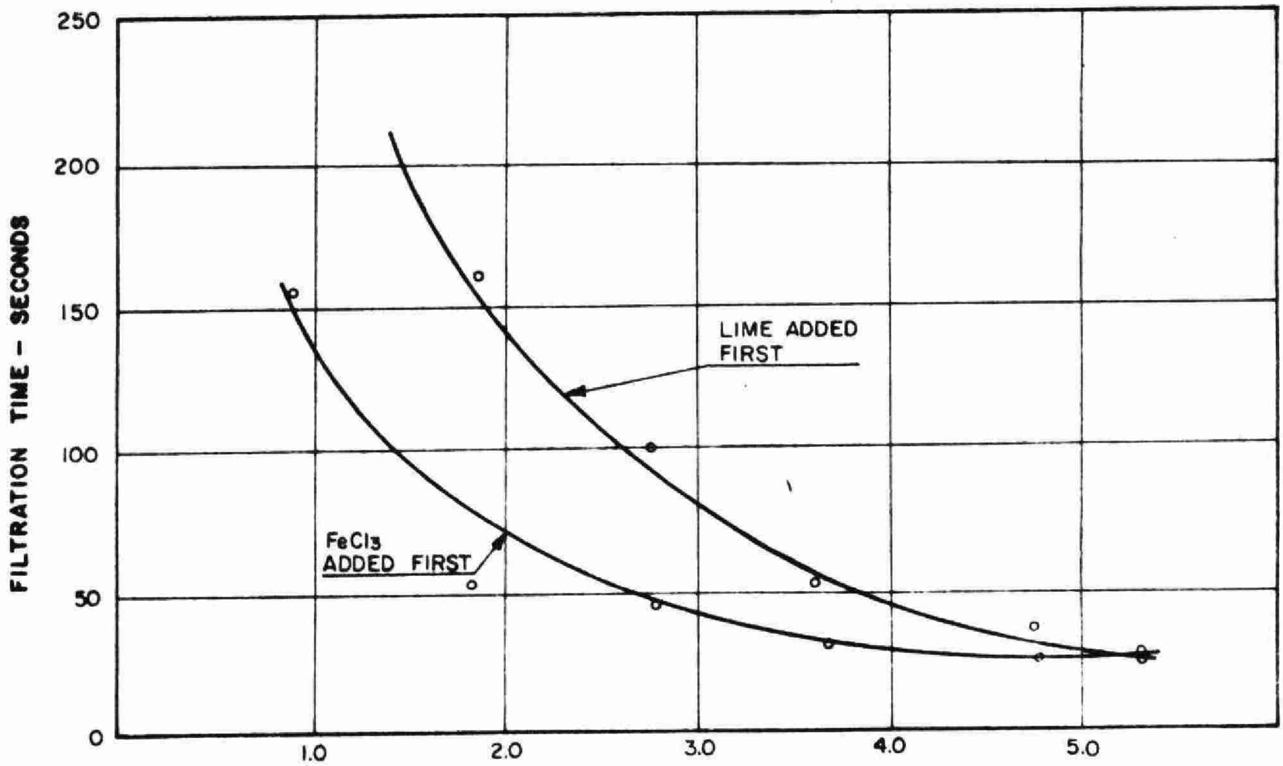
Pilot plant studies have demonstrated the importance of mixing speed. Experiments have been made with paddle, flat blade turbine, propeller and curved blade turbine type mixers. It was found with all of these various types of mixers that the best flocculation occurred during a specific range of mixing speeds. The maximum flocculation was found to occur with the propeller and

FIGURE 3

EFFECT OF SEQUENCE OF CHEMICAL ADDITIONS



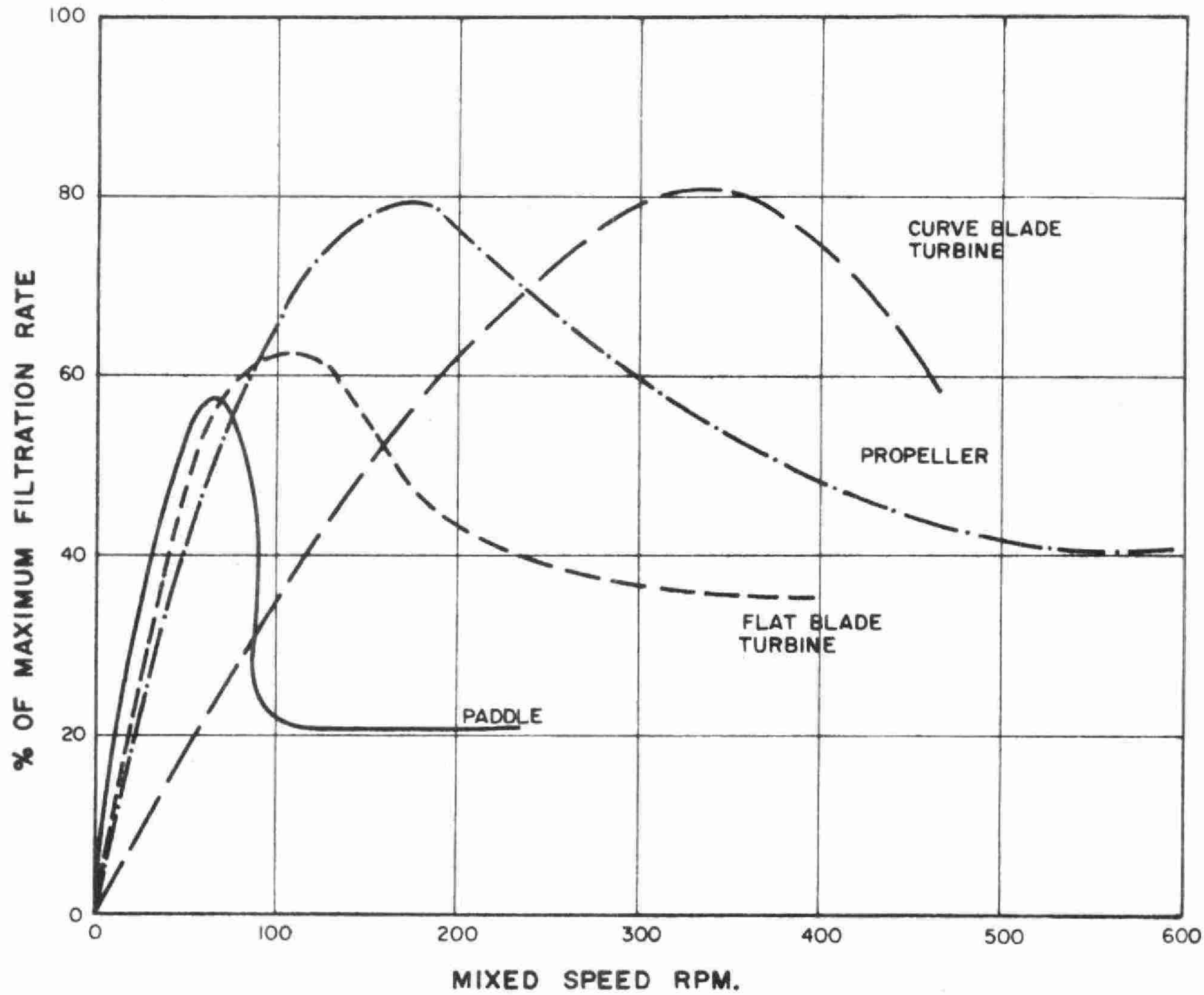
1) LIME REQUIREMENTS



2) $FeCl_3$ REQUIREMENTS

FIGURE 4

EFFECT OF MIXING SPEED ON FILTRATION



curved blade turbine type mixers. The paddle type mixers had a very narrow range of speeds for best results and flocculation fell off quite sharply with speeds on either side of this range (see Figure 4).

Sludges encountered from plant to plant will have different requirements for mixing speeds. In fact, in some plants the characteristics of the sludge vary from day to day. For this reason, it is advisable to be able to change the mixing speed to adapt it to the particular type of sludge being filtered.

Similarly, it is important to experiment with the use of the agitator. Excessive use of this equipment may break up the floc with certain sludges. Many have found only one or two minutes of operation every one-half hour is sufficient.

4. Cake Thickness and Drying Time.

Increasing the cake thickness also increases the moisture in the filter cake. An increase in the drying time (decrease in yield) results in a decrease in the moisture of the filter cake.

5. Quantity of Chemicals

Underdosing of a sludge with coagulants will result in an incompletely coagulated sludge. However, data gathered from several operating filter installations indicate that filter yield can be depressed by over-dosage of chemicals. Since chemical cost is often the largest single item in the cost of filter operation, unnecessary over-dosage especially with ferric chloride can be unduly expensive.

OPERATING RESULTS

Some experiments have been carried out at one of

our activated sludge plants with pickle liquor and ferric sulphate as replacements for ferric chloride. This type of sludge, without elutriation, is one of the most difficult to coagulate and filter.

It was found with both the ferric sulphate and pickle liquor that it was difficult to maintain a good floc. The floc produced was quite unstable and, as a result, the filtrate contained quite a high amount of solids. In fact, it was calculated from laboratory determinations on the filtrate, filter cake and incoming sludge that as much as 50 percent of the solids were being returned in the filtrate.

These solids will settle readily in the primary clarifiers and, hence, cause no problems there. However, this build up of sludge will increase filtering time, amount of chemicals required and thus increase the cost of filtering.

The formulae listed below can be used to determine the amount of solids being returned in the filtrate. It should be noted that it is necessary to know the volume of sludge being filtered and the moisture content of the incoming sludge, filter cake and filtrate before the volume of filtrate and amount of solids can be determined.

$$Vf = \frac{(Cb - Cs)\ Vs}{(Cb - Cf)}$$

$$Bs = \frac{Cb\ Vs}{10} \quad \frac{(Cs - Cf)}{Cb - Cf}$$

$$Fs = \frac{Vf\ Cf}{10} = \frac{Vs\ Cf}{10} \quad \frac{(Cb - Cs)}{(Cb - Cf)}$$

$$Vb = \frac{Bs\ 100}{Cb} = Vs\ \frac{(Cs - Cf)}{(Cb - Cf)}$$

where Vf = volume of filtrate (gals)

Vs = volume of sludge to be filtered (gals)

Cc = % solids in sludge on belt.

Cs = % solids in sludge to be filtered.

Cf = % solids in filtrate.

Bs = lbs. of solids on belt.

Fs = lbs. of solids in filtrate.

Vb = volume of sludge on belt (gals)

The filtering rates for various sludges are indicated below:

Type of Sludge	Average Yield psf/hr.	Range
Primary	10	8.5 - 11.5
Primary digested	8.6	4.5 - 12.5
Primary-digested-elutriated	8.3	3.2 - 13.0
Primary and Activated	3.0	
Primary and Activated-digested	2.7	2.3 - 3.2
Primary and Activated-digested-elutriated	3.1	

The chemical dosages involved in the filtering of sludges vary. They depend primarily upon the alkalinity of the sludge and the organic to fixed solids ratio. Some of the expected dosages for various sludges are shown in the following table:

Type of Sludge	Fresh Solids		Digested		Elutriated Digested	
	$FeCl_3$	CaO	$FeCl_3$	CaO	$FeCl_3$	CaO
Primary	1-2	6-8	1.5-3.5	6-10	2-4	
Primary & trickling filter	2-3	6-8	1.5-3.5	6-10	2-4	
Primary & Activated	1.5-2.5	1.5-4	1.5-4	6-12	2-4	
Activated alone	4-6					

The costs involved in the operation of vacuum filters include labour, power, water, chemicals and maintenance and repair. The costs indicated below do not include repair since the particular plants involved are quite new and repairs have been quite small. Only one plant in each category has been used in determining operating costs. It should be noted that these units have been in operation for only a short time and, as a result, the operating costs may be higher or lower than those that will be experienced in the future.

Type of Sludge	Cost Per ton of Dry Solids
Primary-digested	\$ 7.21
Primary & Activated	19.70
Primary & Activated-Digested	14.10

SUMMARY

Most of the progress with respect to vacuum filtration over the past thirty years has resulted from the operating experience of filter units at sewage plants. Some of the fundamental questions such as an understanding of the mechanisms of conditioning and the reactions involved have only been partially answered. Some work has been done in the theoretical field with respect to specific resistance as a means of expressing filtration results. Carman's theory in this field appears quite sound has agreed with laboratory and full scale plant experiences. Laboratory techniques have been developed for determining the specific resistance of a sludge and the reader is directed to Sewage and Industrial Wastes, Vol. 28, No.8. "Vacuum Sludge Filtration" by P. Coackley and B. R. S. Jones for information with respect to this approach.

The Buchner Funnel Test and the Filter Leaf Test are commonly used laboratory tests for determining chemical dosages. They serve as good indicators but unfortunately do not reflect snall changes in filterability, and, as a result, the best chemical dosages must be determined at the filter unit. Although great advancement has been made in equipment design in the past decade in the dewatering of sludge, more work is needed to determine more effective

coagulating agents which will permit a reduction in costs of filtering. Unless these methods can be found, it appears that other methods of sludge disposal may find their place in the sewage wastes field. The use of coagulating aids and some of the other polymers recently being marketed, as well as the possibility of using wastes from industry as coagulants, may be one method of reducing filtering costs.

The operator of vacuum filters can assist greatly in lowering costs by obtaining the maximum efficiency from his unit. He also should be willing to try new coagulating chemicals as it is difficult at present to correlate exactly between laboratory results and actual full-scale operations.

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DIGESTER OPERATION II

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GENERAL

The first course lecture on the digestion of sludge covered the purpose of digestion, the process, design criteria, and operation. This lecture will concentrate more fully on the operation aspect of the subject. The subject will be covered under three headings:

1. Digester Start Up
2. Single Stage Digestion
3. Two Stage Digestion.

DIGESTER START UP

The following basic procedures can be used in placing a digester in operation:

1. Ensure that all construction has been completed. When the digestion process has been started alterations and repairs or internal parts are difficult to make.

2. Fill all lines and tank with water; raw sewage may be used. Do not use undigested sludge.

3. Add seed material if available. Supernatent or sludge from an operating digester is the only product which will be effective.

4. Heat tank contents to 90 to 95^{o}F and maintain it.

5. Add raw sludge at a rate of .01 pound of solids per cubic foot per day for an unseeded digester and a somewhat higher rate for a well seeded digester. The loading on a high rate process can be .25 pounds of solids per cubic foot per day. The .01 pound of solids per cubic foot per day is equal to 1000 gallons of 5 per cent sludge to a 50,000 cubic foot capacity digester.

6. Circulate the digester contents and maintain the temperature.

7. Periodically check the process by determining the volatile acids and pH. As the process proceeds the quality of the gas may be checked if the testing equipment is available. Also the sludge alkalinity may be determined. Lime may be added to control pH but if the volatile acids advance beyond 2000 ppm the sludge feed is reduced or stopped as required. Please note that excess lime may inhibit the process.

8. Gradually increase raw solids loading on the basis of favourable trends. The loading must not be increased with volatile acids levels above 1000 ppm.

SINGLE STAGE DIGESTION

General

For simplicity the subject, single stage digester operation, will be covered under four headings:

1. Loading
2. Process
3. Supernatent Selection
4. Digested Sludge Removal.

1. Loading

Ideal conditions would be met if the raw sludge could be pumped continuously to the digester. For various reasons continuous loading is not possible. Small plants receiving eight hours per day of operator supervision may load the digester three times a day, say at eight in the morning, 12 noon, and four in the afternoon. Where automatic pumping facilities are provided the other extreme may be reached with loading being effected once each hour. Where supervision is provided on a 24-hour basis, manual control may dictate six to eight pumping cycles per day. Excess water will be directed to the digester if too many pumping cycles are provided. When raw sludge must be pumped for some distance, to the digester, the sludge line must be filled with dilute sludge before the pump is stopped. The next pumping cycle will direct the dilute sludge to the digester.

In a single stage operation the raw sludge is directed to the top half of the digester. As indicated in the flow diagram appended as Item 1, the raw sludge may be mixed with seed sludge leaving the heat exchanger.

2. Process

To maintain the process, two main operating criteria must be met: (a) Sufficient mixing must be afforded to bring the raw sludge in contact with seed material and also to maintain sufficient area free for the digestion process. Where mechanical or gas recirculation equipment is not available a careful check must be kept on the process to ensure that a foaming condition is not created or that the reaction space left does not become too small. In a single stage unit, mixing facilities, if any, are designed to only mix the material in the top half of the tank. In practise such a design is near impossible. Thus it is difficult to obtain a concentrated sludge from a single stage digester operation.

(b) The second process criteria that must be considered is temperature. The ideal operating temperature for mesophilic digestion is between 90 - 95^{o}F. A lower temperature may be used if excess digester capacity is available. Where mixing is not afforded by mechanical means or gas recirculation it is wise to maintain a considerable safety factor to allow for scum blanket space losses.

To maintain a check on the process, various tests and records are required. The number of tests required or that can be economically performed at a plant will greatly depend on the equipment available and the size of the plant. Also where good mixing is afforded the chance of process failure is less; and therefore fewer tests would be required. A few of the tests, listed in order of importance, are as follows:

1. volatile acids,
2. temperature,
3. scum blanket depth,
4. digested sludge depth,
5. supernantent suspended solids,
6. pH,
7. alkalinity,
8. gas compositions,
9. raw and treated sludge composition, volatile and total solids.

Records can also be kept of:

sludge directed to digester,
sludge removed from digester,
quantity of gas,
and mixer operating schedule.

3. Supernatent Selection

It is difficult to obtain a good supernatent from a single stage digester. Nevertheless, an attempt should be made to remove at least some of the excess liquid. Where mechanical mixing is practised the mixing devices are shut off for a period before the supernatent is withdrawn. Experience will show the quiescent period required to obtain a good supernatent.

When a variable level supernatent selection is provided the supernatent is removed via the line proving to be most satisfactory. An example of a supernatent selector system is appended as Item 3. Withdrawal control is maintained in simpler installations by a sleeve height adjustment. Other installations use valves to control the withdrawal process. In all installations a safety overflow should be provided.

The suspended solids test is used to check on the efficiency of the withdrawal process. The actual test can be determined using a centrifuge for quick results, and the standard suspended solids test where complete laboratory equipment is available. The raw sludge directed to the digester may have a suspended solids concentration of 30 to 60 thousand ppm. Therefore, the supernatent suspended solids concentration should not be allowed to approach this figure or little headway will be made in the overall plant operation. A suspended solids concentrations of 1000 to 3000 might be considered permissible with the ideal level being below 500 ppm.

4. Digested Sludge Removal

The accumulated sludge should be removed as frequently as possible. As indicated previously, it is difficult to obtain a good supernatent from a single stage operation. Also it will be difficult to obtain a concentrated sludge from a single stage operation. A four per cent solids content may be considered good for the digested sludge obtained from an activated sludge plant utilizing a single stage digestion process.

When the gas is utilized from a fixed cover operation the digested sludge is best removed when the raw sludge is being pumped. This practise will assist in maintaining the gas pressure and prevent the intake of air which could create an explosive air-gas mixture. A large sludge withdrawal at one time could cause process failure due to lack of seed material.

Bottom withdrawal and depth samples are tested to control the sludge withdrawal process. The total solids and volatile solids tests are two criteria used to evaluate the operation.

TWO STAGE DIGESTION

General

The subject two stage digestion is covered under five headings:

1. Loading
2. Process
3. Sludge Transfer
4. Supernatent Selection
5. Digested Sludge Removal.

1. Loading

When high rate complete mixing is practised the raw sludge may be directed to any point in the first stage tank. Otherwise the loading procedure is similar to that used for the single stage operation.

A good two stage design will allow the use of either tank for the first stage or heated unit. An example of a two stage digester flow diagram is given as appended Item II.

2. Process

Where mixing devices are available they are operated to control scum blankets and inactive dead spaces. Most of the mixing is effected in the first stage tanks. Often mixing units are not installed in the second stage tank. The mixing devices may be operated either full or part time. When part time operation is desired the cycle is set up in relation to

tests and observations of scum blanket formation and not on power saving. In some operations the mixers may only be used a few hours a day.

An improper mixing program could cause a process failure. The active volume available for the digestion process can be greatly reduced by the formation of a scum blanket and sludge banks. Foaming can occur when the scum blanket begins to digest. The scum blanket may be partly controlled by the use of compressed air to mix the tank contents. When using air for mixing great care must be taken to ensure that the explosive air-gas mixture is not ignited. This control measure may be effected two or three times a year depending on need. If just once the designers had to remove the scum blanket from a digester using fire hoses, shovels, etc., they would think twice before designing a digester lacking positive mixing devices. Please be careful when using air to mix the digester content. Forbid smoking in plant area, use rubber footwear, and do not bang metal tools or pipe so as to cause a spark at digester openings. Also, open as many manholes as possible for ventilation.

Heating units are used to heat the contents of the first stage digestion tank. Optimum mesophilic digestion is carried out at between 90 and 95°F. However, lower temperatures may be used where excess digester capacity is available. The maintained temperature should be such as to provide some safety factor.

3. Sludge Transfer

Sludge can be transferred from the first stage digester by a number of means, three of which are as follows:

(a) Automatic transfer may be effected via an equalizing line, as shown on appended flow diagram Item II,

(b) Sludge may be transferred via the heat exchange unit recirculating line.

(c) Bottom sludge may be pumped to the second stage unit.

The transfer program should be set up to delay the removal of solids from the first stage unit. If possible top material is transferred when the mixing devices are off.

Nevertheless frequent transfers must be made from the bottom of the first stage tanks. If this is not done the bottom withdrawal line will plug with grit and solids.

4. Supernatent Selection

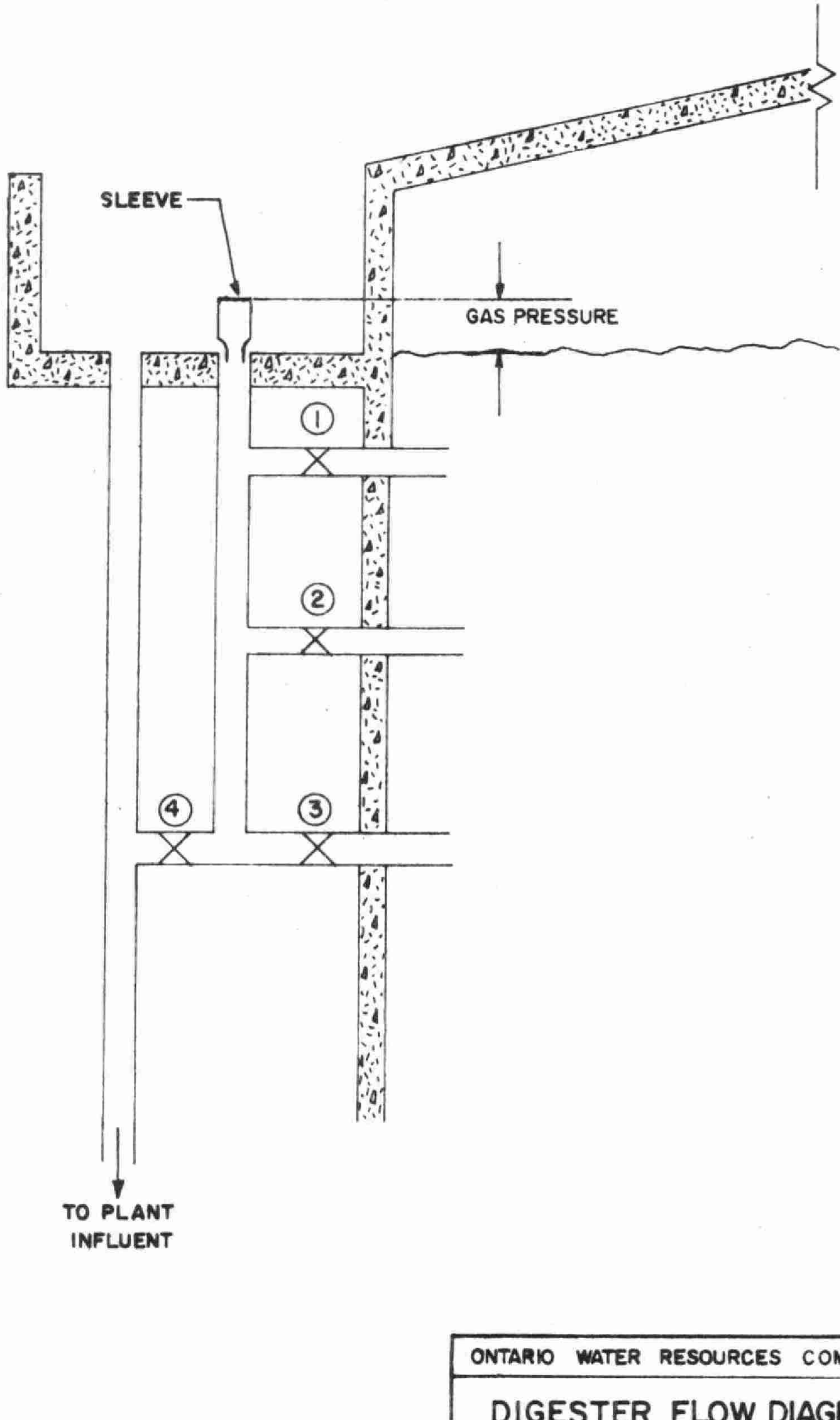
The supernatent is obtained from the second stage digester. The supernatent can be selected automatically when a sludge transfer takes place or as an operating procedure when the plant can best receive the extra BOD loading. The type of selectors provided will of a necessity partly dictate the program to be chosen.

5. Digested Sludge Removal

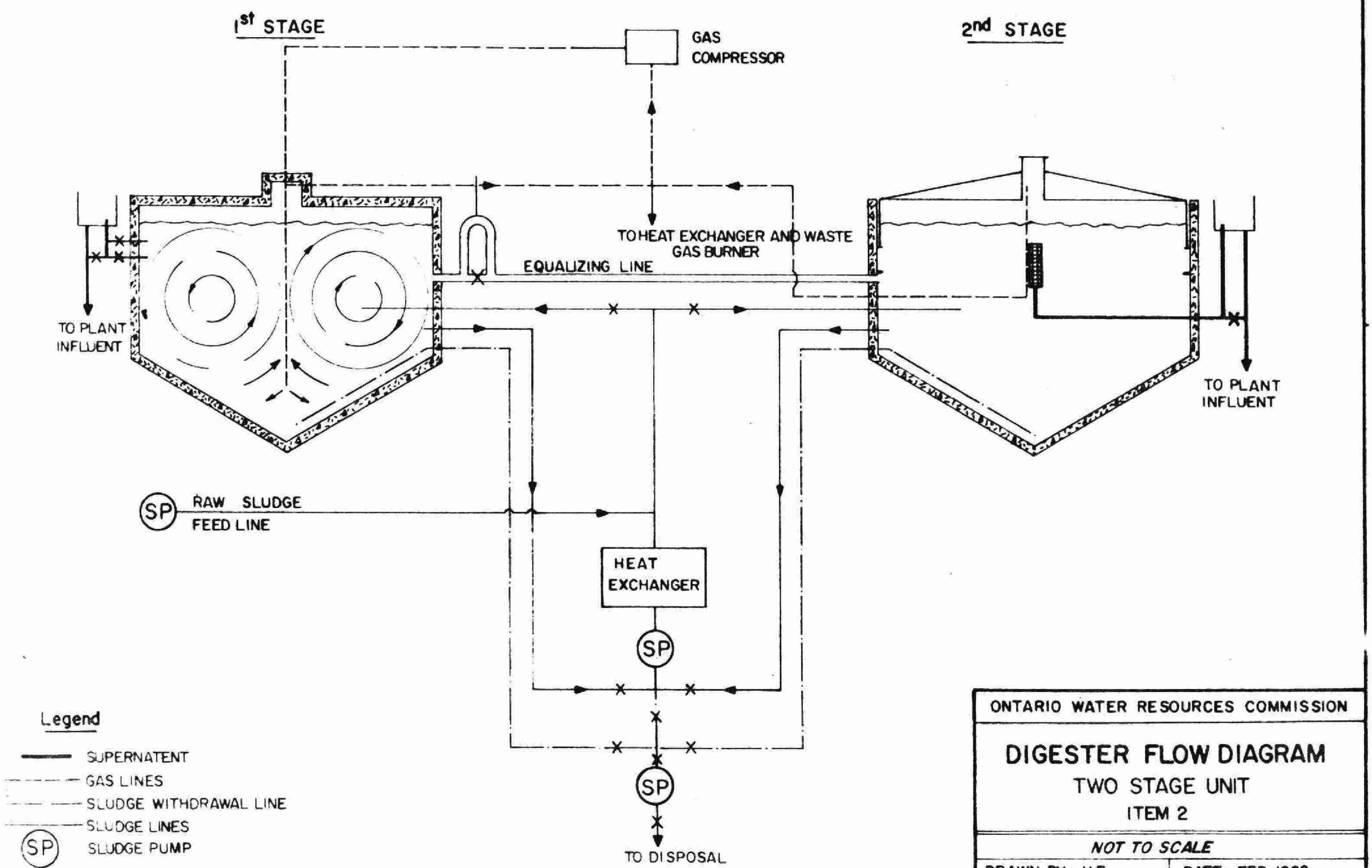
In a fixed cover installation the sludge must be removed in small batches. If this is not done the gas pressure will not be maintained.

When at least one floating cover is provided the sludge settled in the second stage unit may be removed as convenience requires; large withdrawals will not cause process failure or a loss of gas pressure.

Sludge samples should be collected as indicated for the single stage operation. A two stage operation should provide a more concentrated sludge.



ONTARIO WATER RESOURCES COMMISSION	
DIGESTER FLOW DIAGRAM SUPERNATENT SELECTOR ITEM 3	
NOT TO SCALE	
DRAWN BY: H.E.	DATE: FEB. 1962
CHECKED BY: GN	DRAWING No 62-18



ONTARIO WATER RESOURCES COMMISSION

DIGESTER FLOW DIAGRAM

TWO STAGE UNIT

ITEM 2

NOT TO SCALE

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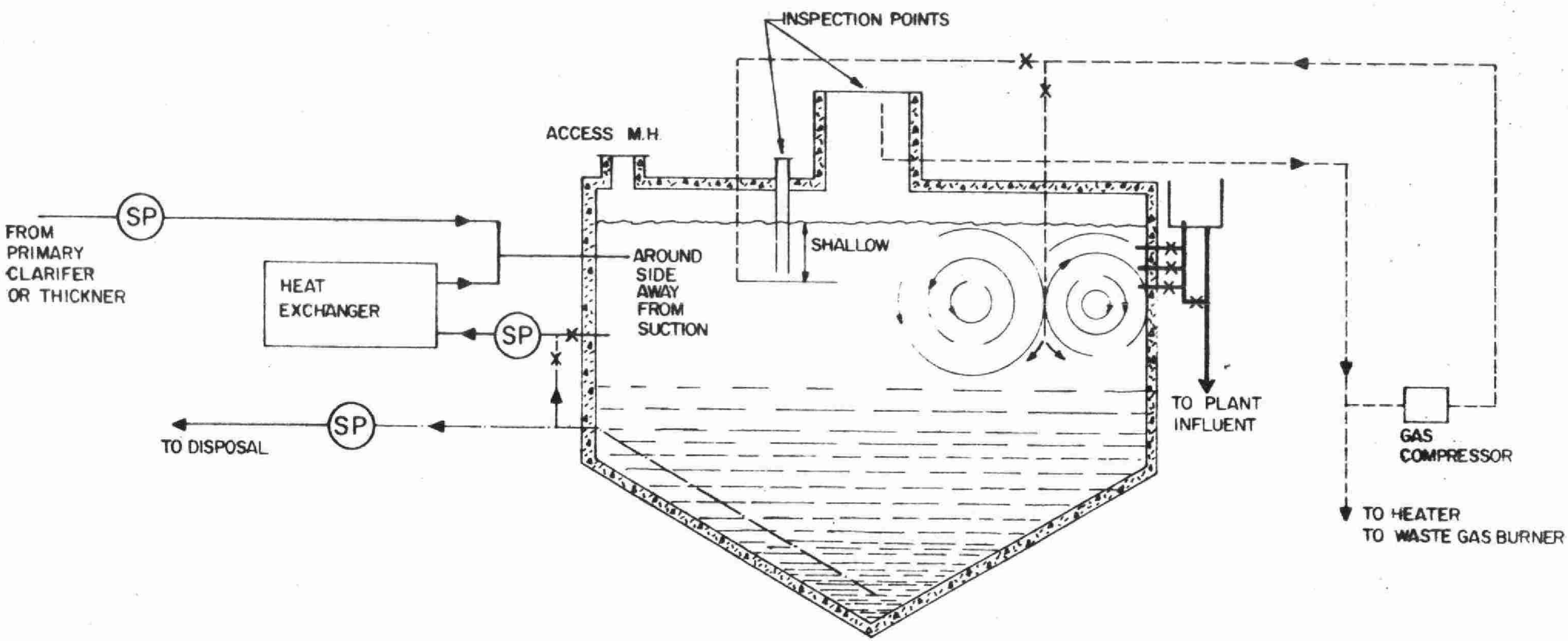


Legend

SUPERNATENT
GAS LINES
SLUDGE WITHDRAWAL LINE
SLUDGE LINES
SP SLUDGE PUMP

ONTARIO WATER RESOURCES COMMISSION

DIGESTER FLOW DIAGRAM

SINGLE STAGE UNIT

ITEM I

NOT TO SCALE

DRAWN BY: H.E. DATE: FEB. 1962

CHECKED BY: CN DRAWING No: 62-16

LABORATORY DEMONSTRATION

THE CENTRIFUGE TEST AND ITS APPLICATION

A. R. Townshend and R. J. Norton

INTRODUCTION

Considerable lecture time has been spent in the presentation of loading factors and formulae associated with the activated sludge process and its modifications which require determination of suspended solids in various sludges. The centrifuge test has been referred to as an acceptable, rapid method of estimating suspended solids.

This test is not a new concept in activated sludge treatment laboratory control. Mr. T. R. Haseltine suggested the use of an electrically operated centrifuge, provided with 15 ml tubes, for quickly indicating the amount of suspended material in the mixed liquor as early as 1937.

In recent years very little has been published to supplement the work done by Haseltine. In Ontario, the Chicago Pump Company has drawn this test to the attention of some plant operators by publishing a procedure in the operating manual it supplied to plants equipped with its mechanical aerators.

The purpose of this laboratory demonstration is to make available to all operators the procedure for conducting the centrifuge test; to correlate the results with the standard suspended solids test which requires time, filtering, heating and weighing equipment; and to show that the results are dependent upon the Sludge Volume Index.

PROCEDURE

The centrifuge comes with two 15 ml. graduated tubes. Both tubes should be filled to the mark with a well-mixed sample from the aeration tanks and inserted in the holders. For uniform results the centrifuge must always be operated at the same speed for the same length of time.

Speeds in the order of 1,500 to 1,800 r.p.m. are commonly used. For hand operated machines this corresponds to about two revolutions per second. Generally, the time of operation used is three or five minutes.

The centrifuge should be allowed to stop without breaking. The volume of solids contained in the bottom of each 15 ml. tube is then read. The average value should be recorded as the result of the test.

Care should be taken to fill the 15 ml. tubes in the same manner for every test. It is considered better to dip the tubes in a well-mixed sample of mixed liquor rather than to pour the sample into the tubes.

Also, the sample of mixed liquor should be collected from the same point in the aeration tanks each day. It is usually taken from the effluent chamber of the aeration tanks as the mixed liquor enters the final settling tanks.

The test should be conducted at least daily. Since the suspended solids change during the day with sewage loading and hydraulic loading, centrifuge tests should be made at the same time each day.

Laboratory glassware and sampling bottles should be cleaned thoroughly with soap and warm water after use.

CONVERSION CHART

Before this test can be intelligently used for control purposes it is necessary to prepare a centrifuge vs. suspended solids conversion curve.

The operator must collect a number of duplicate mixed liquor samples at different times under different loading conditions and determine the suspended solids content by weight and volume (centrifuge test). If facilities are not available at the plant to determine the suspended solids by weight, the duplicate samples may be sent to the OWRC laboratory for analysis.

The two test results for each sample are then plotted on graph paper and a line drawn to fit the points. The graph can then be used for obtaining the weight of suspended solids from the centrifuge test.

Typical conversion curves are shown in Figure I. It may be seen that the slope and equation for these curves vary from plant to plant.

RELATION TO THE SLUDGE VOLUME INDEX (SVI)

The Mohlman Sludge Volume Index is used to indicate the density and condition of the mixed liquor. It is determined by the following formula:

$$\text{SVI} = \frac{\%\ \text{settleable solids} \times 10{,}000}{\text{ppm Suspended Solids}}$$

Both the 30-minute settling test and the centrifuge test are volume measurements. It is reasonable to assume that a light sludge will give a high centrifuge reading as well as a high 30-minute settling test. This suggests that some of the variation found from plant to plant and from time to time in each individual plant may be overcome if the centrifuge test is also related to the Sludge Volume Index.

This concept developed in 1937 by T. R. Haseltine appears to have been lost in subsequent years. A typical diagram for determining suspended solids in the mixed liquor based on the centrifuge test and the Sludge Volume Index as reported by T. R. Haseltine is shown in Figure 2.

DETERMINING MIXED LIQUOR SUSPENDED SOLIDS CONSIDERING SVI

At first, it appears as if the operator must know the suspended solids to calculate the Sludge Volume Index to determine the suspended solids. However, the graph can be solved by trial and error since the 30-minute settling test can be performed by the operator as he does the centrifuge test.

Example I

Assume: (a) 30-minute settling test = 25%
(b) Centrifuge solids = 0.6 ml.

(i) From 45 and less SVI Curve (Fig. 2)

ppm = 2800

Therefore $\text{SVI} = \dfrac{25}{2800} \times 10{,}000 = 90$ which is greater than that assumed.

(ii) From 60 to 100 SVI Curve

ppm = 2300

Therefore $\text{SVI} = \dfrac{25 \times 10{,}000}{2300} = 105$ which is only slightly greater than that assumed.

(iii) From 170 to 210 SVI Curve

ppm = 1850

Therefore $\text{SVI} = \dfrac{25 \times 10{,}000}{1850} = 135$ which is less than that assumed.

It may therefore be concluded that the suspended solids concentration is closer to 2300 than 1850 and may be taken as 2150 ppm for a SVI of 116.

Example II

Assume: (a) 30-minute settling test = 35%
(b) Centrifuge solids = 0.7 ml.

(i) From 45 and less SVI Curve (Fig. 2)
ppm = 3350
Therefore $SVI = \frac{35 \times 10{,}000}{3350}$ = 105 which is greater than that assumed.

(ii) From 60 to 100 SVI Curve
ppm = 2650
Therefore $SVI = \frac{35 \times 10{,}000}{2650}$ = 132 which is still greater than that assumed.

(iii) From 170 to 210 SVI Curve
ppm = 2175
Therefore $SVI = \frac{35 \times 10{,}000}{2175}$ = 160 which is only slightly less than that assumed.

It may therefore be concluded that the suspended solids concentration is about 2300 ppm for a SVI of 153.

For the examples given, by introducing the Sludge Volume Index the concentration of suspended solids only increased from 2150 ppm to 2300 ppm (150 ppm) for a corresponding increase of 0.10 ml by the centrifuge test.

From the graphs shown on Figure I an increase in centrifuge solids from 0.6 ml to 0.7 ml would indicate an increase in suspended solids at Fergus, Richmond Hill and Waterloo of 600, 220 and 150 ppm, respectively.

Operators are encouraged to establish Sludge Volume Index conversion charts at their own plants so they can take advantage of this refinement in the application of the centrifuge test.

SUMMARY

The use of an electrically operated centrifuge to indicate the amount of suspended material in activated sludge mixed liquor has been demonstrated.

Conversion Charts based on (a) Centrifuge tests and (b) Centrifuge tests and Sludge Volume Index have been presented.

An attempt has been made to show that the latter chart corrects for some of the variations that occur with this procedure.

CONCLUSIONS

From the data presently available it is not known whether all plants with the same Sludge Volume Index will give the same Centrifuge curve (with respect to the slope and co-ordinates) or whether each plant will have its own curve.

The operators of activated sludge plants in Ontario with centrifuge equipment can greatly increase the limited information presently available by developing SVI - centrifuge curves for their plants and submitting copies to the demonstrators for study.

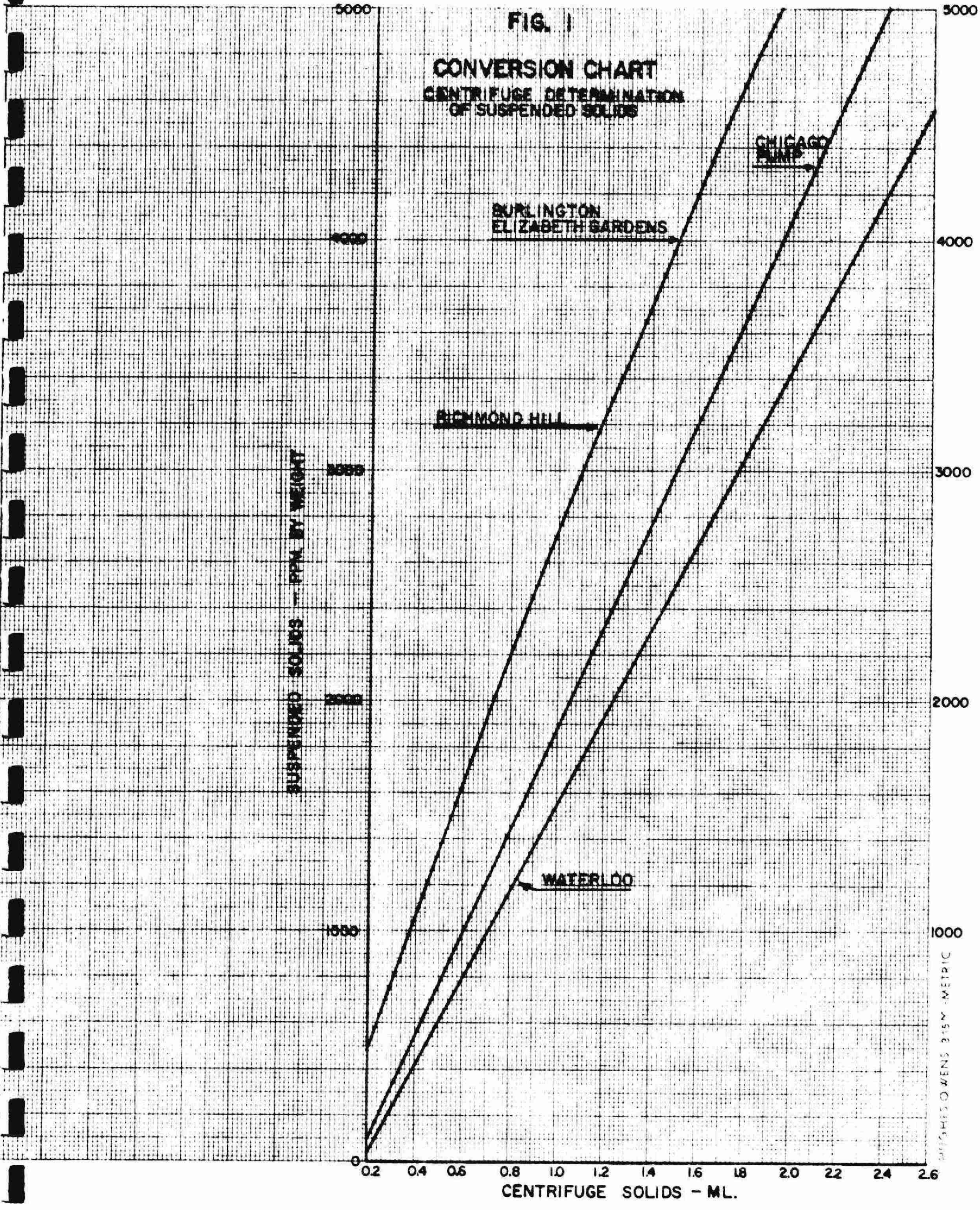
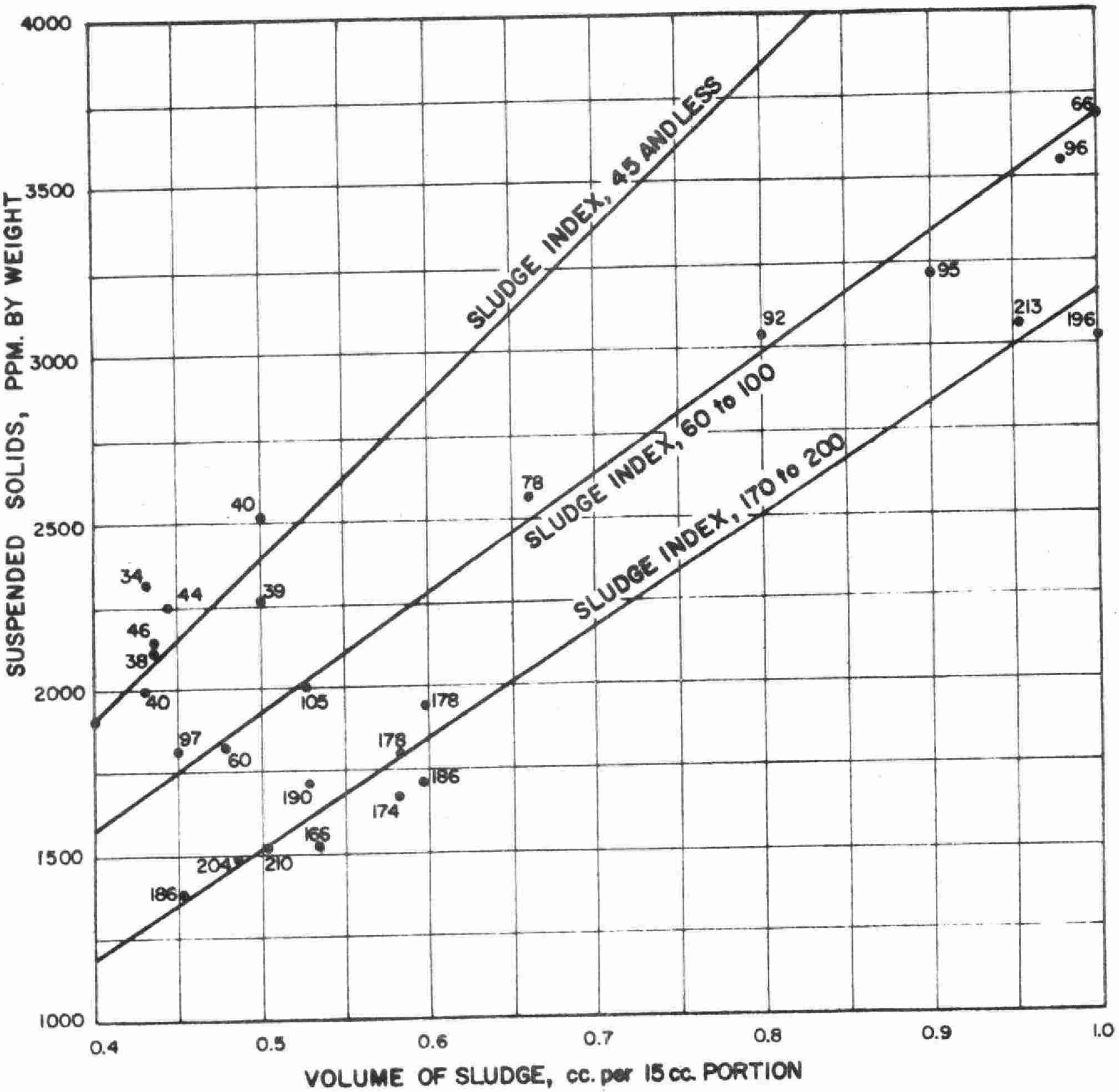


FIG. 1
CONVERSION CHART
CENTRIFUGE DETERMINATION
OF SUSPENDED SOLIDS
CHICAGO
PUMP
BURLINGTON
ELIZABETH GARDENS
RICHMOND HILL
WATERLOO
SUSPENDED SOLIDS — PPM. BY WEIGHT
CENTRIFUGE SOLIDS — ML.
5000
4000
3000
2000
1000
0
0.2
0.4
0.6
0.8
1.0
1.2
1.4
1.6
1.8
2.0
2.2
2.4
2.6

FIGURE 2

TYPICAL DIAGRAM FOR DETERMINING SUSPENDED SOLIDS IN THE MIXED LIQUOR IN AERATION TANKS



LABORATORY DEMONSTRATION

ANALYTICAL METHODS FOR DISSOLVED OXYGEN

C. Howes
Division of Research

Two methods are in general use , the Winkler and the Miller methods. For those intending to carry out either of these tests for dissolved oxygen we strongly suggest acquiring one of the texts previously recommended. These include "Standard Methods for the Examination of Water and Wastewater" Eleventh Edition, and "Analysis of Water and Sewage" by Theroux, Eldridge, and Mallmann, both of which contain detailed instructions for the Winkler test and its modifications.

A sheet of brief instructions for the Miller method was distributed at the demonstration. This is presently under review and the improved copies will be available on request during the final course of this series.

This written summary will therefore include only those points covered during the demonstration which are thought to supplement the above texts.

The Winkler method is the more accurate, with modifications developed to overcome many of the interferences commonly encountered in dissolved oxygen measurement in sewage. To yield the accuracy of which the method is capable, special precautions have to be taken in sampling to avoid exposure to air which can easily upset the original dissolved oxygen content in the sample. A sampling device which provides for flushing of the sample bottle and its sealing under water should be used. Plans for a sampling device of this type are included in both of the above texts. These provide a sample which has not contacted air, and which can be retained in this condition.

The basic chemical reactions involved in the Winkler test are as follows. The manganous compound added, reacts under alkaline conditions with the dissolved oxygen that is present to form a manganic compound. Thus, the dissolved oxygen is trapped as a manganic precipitate. Upon the addition of acid, this manganic precipitate reacts with the potassium iodide previously added, releasing an equivalent amount of free iodine. The manganic compound is converted back into its original manganous form. The released iodine thus traps the original dissolved oxygen in a stable measurable form. The amount of iodine is then measured by titration with thio-sulphate using starch to indicate the final stage of the end point.

These reactions are oxidizing and reducing reactions and if any other oxidizing or reducing materials are present they may react at any stage in any of the reactions to 'interfere', or cause low or high results. It is good practice to titrate the iodine immediately, since the longer this stands the more possibility there is of interference at this stage.

INTERFERENCES COMMON IN SEWAGE

Oxidizing substances interfere in the same manner as would extra oxygen. That is, they give high results. Interferences of this type include exposure to air at any stage in the reactions up to the formation of the iodine. Chlorine is an oxidizing compound and reacts to release additional iodine giving high results. A number of other oxidizing materials may occur in sewage and these may interfere with this test. An oxidizing substance which is commonly present in well treated effluents and in streams is nitrite.

Reducing substances on the other hand tend to react as if there had been a loss of oxygen and thus give low results. The material present in sewage which gives the greatest problem in this respect is organic solids, especially the suspended organic solids. These

may react either with dissolved oxygen in the presence of alkali or they may react with the released iodine at the acid stage. In either case, they give low results. Another possible reducing substance found in sewage is sulphides.

METHODS TO OVERCOME INTERFERENCES

Exposure to Air

This is overcome by proper sampling techniques and by the deft addition of reagents below the surface followed by an immediate careful sealing of the sample bottle. Air bubbles must be excluded from the sample bottle until the iodine stage is reached.

Chlorine

There is a modification of the Winkler test for use in the presence of chlorine but it is difficult and somewhat unreliable. The best solution is to avoid, wherever possible, taking samples which contain chlorine, that is, to sample either ahead of, or some distance below the chlorine contact chamber.

Nitrites

The Alsterburg modification of the Winkler method, the one commonly used, is designed to overcome nitrite interference. The Azide used in this modification reacts with and breaks down nitrites at the acid stage. It also overcomes interference due to small quantities of ferrous iron.

Sulphides

This is not too much of a problem since sulphides and dissolved oxygen are basically incompatible.

That is, they react with one another until one of them is completely used up. Thus if there is a sulphide smell in the sample it is unlikely that dissolved oxygen is present. Rather than proceed with the prescribed modification which is the same one that is used for chlorine it might be best to check whether dissolved oxygen is present merely by the addition of methylene blue either to the sample bottle or to the plant at the point of sampling. If the methylene blue retains its blue colour it indicates that some dissolved oxygen is present. If the methylene blue turns colourless it confirms the absence of dissolved oxygen.

Organic Solids

First of all, do not delay the titration since the free iodine can be used by by these organic solids. A number of methods or modifications have been worked out to try and overcome organic solids interference. The simplest of these is merely to proceed quickly with each stage of the test without waiting for the manganic precipitate to settle out. Where samples are very turbid this alone will not be sufficient. The best approach in these cases is to try and remove the suspended organic solids. This may be possible by settling alone but it is common to use a flocculating agent to weight the suspended particles and carry them more quickly to the bottom. The clear supernatant can then be siphoned off. Copper sulphate solutions are often used and have an added advantage in the following case.

Aeration Tank Liquors

The problem here is that the biological activity in these tanks is so great that during the interval between the taking of the sample and the completion of the test reactions, a good portion of the oxygen present at the instant of sampling may have been used up. A copper sulphate solution added to the

sampling bottle before the sample is taken will instantly mix with it. The copper will kill the bacteria and cause the biological activity to cease. Sulphamic acid may be added as well, since it is both a bactericide and also acts to destroy nitrites. The copper sulphate reacts with the alkalinity in the water and forms a copper hydroxide floc and this carries the particles down leaving a clear supernatant.

To Fix Samples

The copper sulphate treatment above can be used, or for complete reliability when it is impossible to test the samples immediately a combination of sulphuric acid and sodium azide can be used. This completely arrests bacterial action and prevents nitrite interference. When sulphuric acid is used a compensating additional amount of alkaline azide reagent must be added to the sample when the test is performed.

Caution

The reagents used in this test are extremely corrosive. Protective clothing and safety measuring devices should be used. For instance, if pipettes are employed, bulbs rather than the mouth should be used to draw the reagents up into the pipette. The reagents in the Miller method are not quite so corrosive. The worst is the alkaline tartrate which consists of 12 per cent caustic (lye).

MILLER METHOD

The basic chemical reactions involved in the Miller test are as follows. The ferrous iron added (as ferrous ammonium sulphate) reacts directly with dissolved oxygen in the presence of alkali, using the dissolved oxygen up, and forming the ferric form of iron. This is insoluble in alkali and tends to form a

brown precipitate which would obscure the end point. Tartrate is added and combines with this ferric product to keep it in solution as a colourless soluble complex. As long as dissolved oxygen remains the methylene blue stays coloured. At the point where all the dissolved oxygen is used up this indicator dye turns colourless.

The main use of this method has been in the field for stream surveys where the possibility of interference is less likely. Here the simplicity of this method and the absence of extremely corrosive agents out-weigh its inherently lower accuracy.

This is because this method depends to some degree on the technique of the person using it. Thus one operator might consistently arrive at higher or lower results than another even though both their results would be reproducible. For this reason, it is wise for each person performing the test to standardize his technique after practice and if necessary work out a correcting factor to apply to his results. Another problem is that the ferrous ammonium sulphate reagent may gradually lose its strength. It is not necessary to discard this reagent on this account, since a compensating correction factor may also be worked out by frequent checking of this reagent against standards.

In either of the two cases above, the checking or calibration could be performed as follows. The results of the Miller test could be compared with those of a Winkler test performed on two separate portions of the same clear water sample. Alternatively, a sample of water could be left exposed at a constant temperature and allowed to come to equilibrium with the oxygen in the air. The amount of dissolved oxygen which should be present in the sample can be obtained from tables to be found in texts. The actual results obtained with the Miller method could then be compared with this figure. The correction factor would be worked out as follows. If the Miller result read say, nine, where the Winkler or the theoretical result read eight, it would indicate that the Miller method was reading high.

To correct for this, all subsequent Miller analysis done with these reagents should be multiplied by the factor eight/nine to reduce these results to a true reading. In the case where the Miller method gave low results of seven when eight was expected you would increase subsequent test results by a factor of eight/seven. That is, the factor is the expected reading divided by the observed Miller reading.

Disadvantages of the Miller Method

There are no modifications of this method to compensate for interferences. Good technique and rapid analysis must be used to avoid large errors. It is possible to remove suspended organic solids by the same methods used for the Winkler method. Where approximate results are desired, the method may be quite valuable. In any case, do not accept results without question. Always keep in mind that the results for both these methods may be seriously in error if good techniques are not employed.

MEASURING DISSOLVED OXYGEN BY METERS

The methods of measuring dissolved oxygen by meters will also be illustrated during this lecture.

SLUDGE DISPOSAL BY TRUCK HAULAGE

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Project Services Engineer
Division of Plant Operations

INTRODUCTION

This lecture is intended to supplement the information that has been presented to you in the Basic Sewage Works Course. On many occasions in the future, you may be called upon to give your views of sludge disposal and it is hoped that this lecture will provide some background information. The majority of sewage treatment facilities employ trucks for the final disposal of sludge onto land although in some coastal municipalities sludge is disposed of at sea.

METHODS OF SLUDGE DISPOSAL

Many people, in their description of a treatment facility, have employed phrases such as : "sludge disposal by drying beds." As Kershaw has stated, the use of drying beds is not a method of sludge disposal; it is a stage, and a stage only, in the disposal of sludge.

The various methods (or stages) of digested sludge disposal have been discussed in the Basic Course and are listed below in an approximate order of simplicity and economy. It should be noted that all of the following may require truck haulage for the disposal of the final product.

1. Sludge lagooning of digested sludge only. This method may require the removal of the dried sludge by a tractor or drag line for final disposal by truck at a suitable site. This method is considered to be somewhat obsolete.

2. Sand drying (filter) beds provide a relatively good method of treating sludge for final disposal but usually a fair amount of valuable land is required. After the sludge has dried it must be removed by either plant personnel or other people for final disposal on land.

3. Liquid haulage of digested sludge by tank truck for disposal on land is perhaps the most common method employed at the present time. More will be said on this in the following section.

4. Vacuum dewatering of sludge followed by sludge cake haulage is employed in a number of plants.

5. Vacuum dewatering followed by sludge incineration or drying is employed in certain cases where haulage of the wet cake is too expensive.

6. Wet oxidation methods employing either the Zimmerman Process or the Atomized Suspension Technique have been used in isolated cases mainly as full scale experiments. These processes reduce the organic solids to fly ash but the liquid portion requires further treatment. At the present time it appears that the capital cost for the installation of the above processes is too high to be competitive with the more common methods.

HAULAGE OF LIQUID SLUDGE

Over the past few years, operating authorities including the Division of Plant Operations have determined that the simplest and most economical method of sludge handling, after digestion, is liquid haulage even though other methods such as vacuum filtration are at their disposal. In 1954, the Ministry of Housing and Local Government (Great Britain) categorically stated in a report that the most satisfactory, the most economic, and the most suitable

method of sludge disposal is by direct application of the digested sludge in the liquid state to land areas. Kershaw et al used liquid haulage as an interim measure when their drying bed capacity was overtaxed. They had calculated that liquid haulage would be cheaper than the use of the new drying beds which were then under construction. In their case, the use of liquid haulage avoided overhead costs in maintaining the sludge drying beds and in addition, land and construction costs.

Where a plant is located in a residential area, the designer must consider the problem of odours and other nuisances. The provision of closed digesters plus an efficient method of liquid haulage is a good solution to the problem. It is important that any "Sloppy Joe" approach to liquid haulage be avoided for the purpose of promoting good public relations.

In addition to the above considerations an obvious reason for employing liquid haulage is that no other means of sludge disposal has been provided.

SLUDGE QUANTITIES

It is necessary to know the amount of sludge that can be produced at a sewage treatment plant for design requirements, budget requirements and for process control and efficiency. The volume of sludge that is produced is dependent on a number of factors, three of which are listed below :

1. Raw sewage strength and quantity.
2. Type and degree of treatment provided.
3. Type and degree of sludge treatment.

The sewage strength is normally measured in terms of B.O.D. and suspended solids concentrations. These measurements can be affected by such parameters as the type of industries on the sewage system, the degree of infiltration into sewers, and the consumption of water per capita. For a normal domestic waste, the loading on a plant can be estimated by allowing 0.17 pounds B.O.D. and 0.20 pounds suspended solids per capita per day.

The degree of treatment that a sewage treatment provides will affect the quantity of sludge produced. The following table lists the removals expected at various types of plants :

EXPECTED REMOVALS

Type of Plant	BOD_5	S. S.
Primary Sedimentation	30-60%	50-60%
Single Stage Low Rate Trickling Filter	80-90%	80-90%
Conventional Activated Sludge	85-95%	85-95%

For the purpose of this lecture and without considering the more exotic forms of sludge treatment, we can consider that sludge is either digested or it is not. As a general rule, digestion provides for approximately a 40 to 60% destruction of the volatile solids but this percentage can be higher. This destruction, of course, is dependent on such factors as mixing, temperature, retention time, and the initial volatile solids content of the sludge. The volatile solids content of the sludge can range from 60 to 85%.

The concentration of sludge solids can affect the detention period of the sludge in the digester. This concentration of sludge may vary from primary sludge at 8% solids to waste activated sludge at less than 1% solids. Liquid digested sludge usually has a dry solids content of 3 to 5% whereas vacuum filter cake can have a dry solids content of, say, 18 to 25%.

One should note that in a conventional activated sludge, excess (or waste) sludge is produced and must be disposed of. The quantities generated depend, in part, on a relationship between the oxygen available, the concentration of organisms and the organic load. However, this aspect of sludge production is beyond the scope of this lecture and will not be discussed further.

EXAMPLE CALCULATIONS

Assuming that the following waste characteristics are received at a primary treatment plant calculate sludge quantities at various stages.

Type of waste	-	domestic sewage
Flow	-	2.2 MGD
BOD	-	200 mg/1 (ppm)
S. S.	-	200 mg/1 (ppm)

Assuming 50% removal of suspended solids, we have :

$$\frac{50}{100} \times 200 \text{ ppm} \times 2.2 \text{ MGD} \times 10 \text{ lb/gal}$$

= 2200 lbs dry solids/day removed to the digester.

In terms of a 6% liquid, undigested sludge we have :

$$2200 \text{ lbs} \times \frac{100}{6} \times \frac{1 \text{ gal}}{10 \text{ lbs}}$$

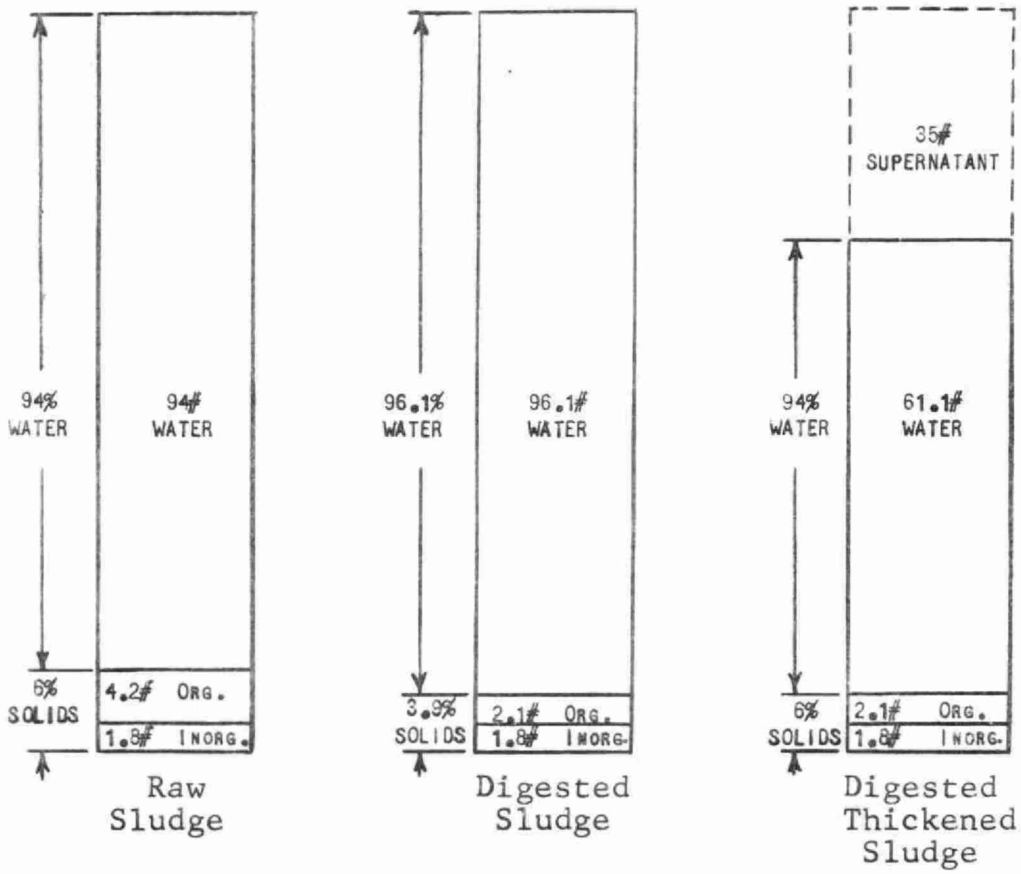
= 3660 gal/day pumped to digester.

In terms of a 6% liquid, digested sludge where the total solids reduction is 35% and the supernatant is decanted off we have :

$$2200 \times \frac{100}{6} \times \frac{1}{10} \times \frac{100 - 35}{100}$$

= 2400 gal/day to be hauled.

The above can be shown in the schematic below where the volatile solids content is 70% and that the volatile destruction is 50%.



In the case where the raw sludge is filtered we have (for a 20% solids content in the cake) :

$$2200 \text{ lbs dry solids} \times \frac{100}{20}$$

= 11,000 lbs of cake to be hauled.

In the case where the digested sludge is filtered we have (for a 20% solids content) :

$$2200 \text{ lbs dry solids} \times \frac{65}{100} \times \frac{100}{20}$$

= 7,150 lbs of cake to be hauled.

VEHICLE DESIGN

In purchasing or designing a sludge haulage vehicle, the following factors should be considered :

1) Quantities of sludge to be hauled.

2) Length of haul (long or short).

3) Type of sludge (liquid or filter cake).

4) Availability of staff or contractors.

5) Other uses for truck
   e. g. Industrial Wastes, Fire Truck, removable tank on stake body.

The Department of Transport has regulations limiting the loadings on vehicles similar to the table below :

	G. V. W.	V. W.	Pay Load
2 axle	28000 lb	13000 lb	1500 I Gal (or 7½ cu. yds)
3 axle	42000 lb	18000 lb	2400 I Gal (or 12 cu. yds)

Generally, four wheel drive is recommended for the two axle units while tandem drive is recommended for the three axle units.

Liquid Sludge

The steel tanks for hauling liquid sludge are usually constructed with an oval or circular cross-section. The loading of these tanks is by means of pumping facilities provided at the plant through an access manhole located on the top.

A four inch discharge valve is considered adequate but special attention is required to prevent freeze up during winter operation. For example, a top operated valve could be used with the valve itself located in the tank itself. The tank should be equipped with adequate internal baffles, inspection ports, and vacuum relief valves.

Selection of Tank Size

From the previous example we have 2400 gal/day of digested sludge to be hauled. Assume that sludge is hauled on the basis of a 4-day week

$$\therefore \quad \frac{2400 \times 7}{4} = 4200 \text{ gals/day to be hauled.}$$

For a short haul, 10 loads, a 420 gallon tank is required. Perhaps it would be better to haul only 2 days each week, therefore an 840 gallon tank is required.

For a long haul, 6 loads, 2 days each week, we require

$$\frac{8400}{6} = 1400 \text{ gallon tank.}$$

The above calculations indicate a need for a 800 to 1400 gallon tank depending on the length of haul. In this case, a two axle, four wheel drive vehicle may be advisable.

Filtered Sludge Cake

Sludge cake can be hauled by dump trucks equipped with a water tight gasket on the rear gate of the box. Portable steel containers such as those used by the industrial waste disposal companies are also quite effective in hauling sludge cake. In either case, the sides should have more than normal free board by allowing one cubic yard of volume for each ton of cake to be carried.

PUBLIC RELATIONS

The sewage treatment plant operator must always do his utmost to avoid any bad publicity with respect to sludge handling. He can promote good publicity by

1) ensuring efficient sludge haulage, and

2) ensuring that suitable disposal sites are available.

It should be noted that some municipalities are reluctant to allow any sludge haulage in their areas, let alone allowing sludge disposal within the area. The restrictive regulations in these municipalities no doubt were passed because of numerous well founded complete.

Liquid raw sludge perhaps has the most obnoxious odour and thus the disposal sites should be isolated. It is important that any "Sloppy Joe" haulage be avoided.

Liquid digested sludge is perhaps the best method of handling sludge in residential areas because it is relatively odourless.

The disposal of sludge cake can cause problems as it is quite difficult to handle in the field. For example, it has to dry for a period of time (sometimes up to a year) before a bulldozer can handle the material.

Some general precautionary measures that promote good public relations are as follows :

1) Care should be exercised in selecting disposal sites. If the driver of the vehicle is entrusted with the job, the disposal sites should be inspected by plant personnel for suitability and the driver should maintain a log on each dumping.

2) Vehicles should be kept as clean as possible.

3) Valves on the liquid disposal units should not leak.

4) Where sludge cake is being handled, the rear gate of the dump box should be equipped with a rubber gasket.

5) Where sludge is removed under contract, the contract documents should include an outline of disposal practices.

PUBLIC HEALTH CONSIDERATIONS

The Department of Health is responsible for promoting public health within the province. With this in mind, the Department prefers the disposal of digested sludge over raw sludge and usually considers the following points in approving a disposal site :

1) Odour

2) Type of crops

3) Parasitic ova and viruses

4) Animal wastes in the area.

Since sewage treatment plant operators are involved in the handling of sludge, perhaps it is best that the above points be discussed.

Digested sludge generally has no odour other than a slight mustiness and is not considered objectionable. If liquid sludge must be stored in a holding lagoon for a period of time, then consideration should be given to the possible problem of odours.

The use of market gardens is not considered advisable as disposal sites. Other types of crops are usually allowed to receive solid wastes. Certain crops will allow the land to be used only at certain periods of the year because of harvesting, trimming, etc.

The degree of survival of disease organisms in digested sludge varies. The only thing that we are sure of is that heat treating of sludge renders the sludge sufficiently safe for soil conditioning purposes. Anaerobic digestion at temperatures less than 100° F will not completely destroy Ascaris eggs. The destruction of pathogenic viruses by digestion is not known and hence, viruses must be considered as a potential hazard. The proper control of land disposal areas can avoid any serious health problem.

The disposal of animal wastes is becoming a problem in certain areas as there is no effective control at present. Competition for the disposal areas may prove to be a problem in future years. It is possible then that the purchase of land disposal areas may be required for future requirements.

SLUDGE AS A FERTILIZER

As sludge must ultimately be disposed of on land areas the sewage treatment plant operator should be aware of the following factors in the promotion of sludge as a fertilizer :

1) Sludge is not the best fertilizer and is usually considered as soil conditioner.

2) Sludge contains small amounts of the principal fertilizer ingredients (nitrogen, potassium and phosphorus).

   Sludge also contains many trace elements and growth promoting substances which may be of value in particular soils.

3) The lignin content of sludge is a factor in increasing the humus content of the soil. Sludge then improves the soil structure in general and increases the water holding capacity of certain soils.

4) Only 40 to 60% of the nitrogen in sludge can be used immediately by plant life but there is a residual supply of nitrogen which

when converted to the proper form prevents the "burning" of lawns and plants.

5) The quantity of nitrogen, phosphorus and potassium present in the sludge is dependent on the type of treatment. For example, undigested activated sludge has the greatest amount of the above elements whereas digested raw sludge has the least amount. A table indicating the typical ranges of the fertilizing ingredients in various organic materials is shown below :

MATERIAL	FERTILIZING INGREDIENT (%)			
	NITROGEN (N)	PHOSPHORIC ACID	POTASH	ORGANIC MATTER
Digested settled sludge	0.8-3.5	0.7-4.0	-	-
Digested settled sludge with trickling filter sludge	1.0-4.5	tr.-4.0	0.8-1.6	30-60
Digested activated sludge	2.0-4.8	1.0-3.6	-	-
Heat dried-activated sludge	4.0-7.0	1.7-2.5	0.13	-
Commercial Pulverized				
Sheep manure	1.2-2.5	1.0-2.0	2.0-4.0	48
Cattle manure	1.6-2.1	1.0	1.0-2.2	66
Poultry manure	1.9-4.0	2.5-3.7	0.8-1.3	64

6) The loading rates of sludge on land disposal area varies from 100,000 to 400,000 gallons of 5% sludge per acre per year. Some authorities feel that a safer limit is approximately 10,000 gallons per acre per year.

Generally, the loading rate depends on the local soil conditions and because of this, it is best to consult the Department of Agriculture or the Ontario Agricultural College. For example, in the St. Catharines area, the recommended rate of application at 5% solids is 150,000 gallons/acre/year. The application of nitrogen should not exceed 150 to 300 pounds/acre/year. It should be noted that there is no nitrate nitrogen in digested sludge.

SUMMARY

1) The calculation of sludge quantities is a basic item which is valuable in checking the operation of your plant and is of prime importance in sludge disposal by haulage.

2) Although many methods are used to handle sludge, the ultimate disposal of sludge is on land employing truck haulage.

3) It is possible that the vehicle design may be your responsibility sometimes in the future. Know what you require in the vehicle before you start and know your governmental regulations as to loadings. Give just consideration to the small features of a vehicle which make it more economical to operate and which have public relations in mind.

4) Keep in mind that good public ralations are perhaps the most valuable asset in a well developed sludge haulage program. Bad publicity is something which could increase the annual budget of your plant by thousands of dollars.

5) Public Health considerations should always be foremost in the operator's mind when selecting sites for land disposal area.

6) The fertilizer value of sludge is existent, however, sludge should be classified primarily as a soil conditioner. In using sludge on farm land, the lack of contact between crops which may be consumed raw and sludge should be stressed.

CONTROL OF INFILTRATION INTO SANITARY SEWERS

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INTRODUCTION

One of the problems encountered in sanitary sewers is "Excess Water". This excess water in a sewage collection system comes from the water in the ground itself and from surface drainage. The effect of this excess water is to create additional work at the treatment plant and subsequently additional costs for the owner. It should also be pointed out that this will result in additional pollution of the receiving waters when it is necessary to by-pass the treatment facilities because of the excess amount of water being delivered to the plant. For sewage treatment plant operators, excess water spells trouble.

One major source of excess water is the practice of connecting house footing drainage (weeping tile) and storm water run-off sewers to the sanitary sewers. This practice is prohibited wherever possible but most older municipalities have what is known as a 'combined system' and the separating into sanitary sewers and storm sewers becomes an expensive proposition. For example the City of Toronto is contemplating a staged sewer program over the next 25 years at an estimated cost of $100 million. Ottawa is staging a plan over 10 years at an estimated cost of $25 million. In each of the two examples given, the separating of the sewers into storm and sanitary is a major expense.

Some smaller municipalities, in order to overcome overloading their sewage treatment facilities, often require storm water to be pumped from each house or business and discharged into open ditches. This is not a desirable procedure.

Every plant operator has witnessed how the volume delivered to the plant increases after a rain storm. This "excessive water" decreases the effectiveness of the treatment and makes the plant and sewers become inadequate in size long before their time.

With the increasing population and the demand for more and more clean water, the time will come when even minor pollution of our waterways will not be permitted. It is often stated that when a plant "by-passes" because of the storm flow, the sewage is well diluted. This may be true at the present time but as population increases the pollution of the receiving waters will also increase.

The treatment plants of today are designed on the basis of handling approximately four times the dry weather flow and if we wish to make the most possible use of our plants, the storm water must not be allowed to enter our sewage collection system.

Since the conception of the OWRC and with particular reference to the Division of Construction, there has been a great deal of emphasis on the elimination of infiltration into sewers. The Commission has developed what may be referred to as "tight" sewer systems. Up until six to ten years ago infiltration up to 1300 Imperial gallons per inch diameter per mile per day was considered an acceptable installation. In fact, a survey taken in the United States a few years ago indicated that out of 500 cities, a third reported infiltration was up to 50 times the allowable of 1300 Imperial gallons. The Commission, on the other hand, has adopted a policy that the maximum infiltration should not exceed 300 I.G./"Ø/m/d.

With more attention being paid to the tolerances in the manufacture of sewer pipe and also the workmanship in connection with the installation of the sewers, the sewer mains being installed today should last for a great number of years.

SOURCES OF INFILTRATION

Infiltrated water comes from several sources other than a direct storm sewer connection. They are

1) Cracked pipe. This pipe may have been installed in a damaged condition or may have failed under the superimposed loads.

2) Improperly plugged service connections.

3) Poorly constructed reinforced concrete manholes or poorly installed pre-cast manholes. The manhole covers themselves. Poorly located manholes such as those installed in a ditch with the top at such a low elevation that the ditch drainage is accepted into the sewer system.

4) Faulty sewer pipe joints.

Dealing with these sources of infiltration in turn:

Cracked Pipe

The installation of damaged pipe is fairly rare since the site inspectors are continually checking for visual defects. In some cases the manufacture of the pipe is suspected when the pipe fails during installation.

Most frequently a cracked pipe is the result of an incorrect class of pipe being chosen for a particular installation. The selection of the class of pipe for a particular installation is the responsibility of the designer. However, it is the responsibility of the site inspector to see that this class of pipe is installed in the particular installation for which it was chosen. In this connection, it is the width of trench which is the dominant factor on the superimposed load the pipe must sustain. For example, a sewer pipe may be chosen for a certain width of trench and during installation the contractor may exceed this width of trench beyond the safety factor for the sewer pipe.

All pipe suppliers issue bulletins recommending the proper class of their pipe for various installations. The structural strength that the pipe must have is determined from the width of the proposed trench, the unit weight of the backfill material and the proposed pipe bedding. It is easier to specify trench width and bedding than to obtain them in the field. The site inspector should always keep these requirements in mind and he should be made aware of the particular relationship between the design and the installation so that if necessary the class of pipe or bedding can be altered to suit the particular installation.

House Service Connections

When new sewage collection systems are installed it is the practice to install the house services from the main sewer in the street to the property line. Many of these house services are installed on speculation for future use. If these unused services are not properly capped they may act as storm drains, draining the ground water from the adjacent area. The site inspectors on Commission projects are requested to inspect each house service installation to be sure that it is capped properly.

Another problem arises when the house service is connected into the main sewer. These junctions are usually made with a pre-formed tee, wye or a saddle. The contractors will often consider that breaking into the main sewer and mortaring in the house service connection is acceptable. We have had a number of occasions where such a connection has not only caused a great deal of infiltration but has allowed the bedding and surrounding granular material to enter the sewer system.

These house service connections should be properly supported and bedded to prevent settlement and eventual cracking of the pipe.

Manholes

With the construction of reinforced concrete manholes, precautions should be taken to ensure that a compact, void free concrete wall is obtained. Many of the faults in pouring concrete walls can be eliminated by the proper vibrating of the concrete during pouring. It is also essential that a proper bond between the floor and the walls be obtained.

The tendency today in the installation of sewage collection systems is the use of the pre-cast manhole sections. These are manufactured similar to the concrete pipe except that the top section is usually sloping and fabricated with a thicker wall to obtain more weight. The sloping sides of the concrete top section and the weight decrease the possibility that this section will be lifted by frost. In some cases where excessive ground

water is anticipated, these pre-cast manhole sections are provided with a rubber gasket at the joint. The contractors themselves prefer the pre-cast concrete manholes because they are faster to install and, therefore, the installation of the manholes can keep pace with the installation of the sewer. In the case of the reinforced concrete manholes, of course, the excavation for the manhole must be left open for a longer period of time to allow for the construction of the forms, the pouring of the concrete and the removal of the forms.

The manhole covers should be properly sized and should contain only two lifting holes. Manholes which occur in ditches or in locations of heavy surface water should be extended to a height well above the anticipated water level. It is good practice also to waterproof this manhole extension in ditch locations to prevent the entrance of any surface water.

Joints

The joints in the sewer main are by far the worst offender as far as infiltration is concerned, mainly because of their number. A sewer pipe joint should be watertight to prevent infiltration and root penetration. They should also be flexible since there will usually be a minor amount of movement of the pipe when it is backfilled.

The first sealant to be used on concrete sewer pipe is a combination of oakum and mortar. The oakum was caulked into the joint followed by the sealing of the outside of the joint with mortar. This type of joint was found to be far from adequate since workmanship played a large part in the effectiveness of the joint being made and the drying mortar cracks and admits ground water. This was also a rigid joint and any minor differential settling caused either the mortar to crack and chip out or the pipe bells to be broken.

With the emphasis on decreased infiltration an asphaltic coal tar base jointing material was developed which has had some excellent results. There were actually two types of asphaltic jointing material; one which had to be heated and poured into the joint, and the other which was caulked into the joint. The quality of the joint made with this type of material depends greatly on the workmanship and in too many cases leakages resulted from improper care being taken.

The sewer joint which is becoming widely used and which is accepted by the Commission is a pre-formed joint using a rubber gasket. At the original development of this rubber gasketed joint it was considered to be the answer against infiltration. However, problems arose with this type of joint in maintaining adequate gasket pressure. This rubber gasket joint pipe was manufactured to tolerances of about 1/16" between pipe surfaces with a 15° slope on the bell and spigot. This slope combined with the gasket pressure had a tendency to push the pipe back out after they were put together. When a pipe is pushed out in this manner, the gasket pressure is relieved and the joint develops leaks. If this type of situation developed say in poor soil conditions such as a fine running sand, the sand and perhaps the bedding will be admitted through the joint into the pipe and will eventually lead to undermining and the possible failure of the pipe.

In order to obtain the most suitable pipe for sewage collection systems, the OWRC has prepared detailed specifications for concrete sewer pipe up to and including 36" in diameter.

The Commission's Specifications for Concrete Sewer Pipe 36" dia. and under, require that the slope of the tongue and the bell be limited to 2°. With this slope the tendency to spring apart when the pipes are pushed together is almost eliminated.

TYPES OF SEWER JOINTS

The OWRC Specifications for Concrete Sewer Pipe are for diameters 36" and less. This should be kept in mind when we are discussing the different types of sewer joints. The specifications prepared by the Commission are

supplementary to the American Society for Testing and Materials Specifications on Concrete Sewer Pipe, Nos. C-14, C-76 and C-443. In cases of conflict between the ASTM Specifications and the OWRC Specifications, the OWRC Specifications will govern.

The Commission has also developed a procedure of prequalification of concrete pipe manufacturers for Commission projects. Only a pipe produced by a supplier who has been prequalified by the OWRC will be allowed on Commission projects. In this way an inspection can be maintained on the suppliers' production procedures. Manufacturers are tested in accordance with the requirements of the prequalification specifications by an independent inspection firm and an inspection report is forwarded to the Commission.

JOINTS

Briefly, the pipe joints recommended for Commission projects are:

(1) The recessed tongue.

(2) The bell and spigot with roll-in gasket.

(3) Single off-set spigot.

(4) The wrap around rubber gasket (external flexible rubber gasket).

(5) The asbestos-cement pipe, rubber ring joint.

(Modified) Recessed Tongue

The recessed tongue concrete pipe shall have a rubber gasket snapped into a recess or groove cast in the pipe tongue. The designed angle of taper on the conical surface of the inside of the bell or groove and the surface of the spigot or tongue measured from a longitudinal trace on the inside surface of the pipe shall not be greater than 2°. Measured slopes may be $2\frac{1}{2}^{\circ}$.

Bell and Spigot Pipe With Roll-in Gasket

The inside surface of the bell and the outside surface of the spigot shall have a designed slope no greater than 2° measured from a longitudinal trace on the inside surface of the pipe. Measured slopes may be $2\frac{1}{2}^{\circ}$. The inner wall of the bell shall have a protruding shoulder so that when the clearance between the shoulder and the spigot is closed, there will still be sufficient gasket pressure to make the joint watertight.

Single Off-Set

The spigot end shall be so formed that a single off-set is formed on the leading end. The surface of the inside of the bell and the outside of the tongue shall be parallel and shall have a maximum designed slope of 2° measured from a longitudinal trace on the inside of the pipe. Measured slopes may be $2\frac{1}{2}^{\circ}$.

External Flexible Rubber Gasket

This pipe shall be of the tongue and groove type. The tongue and groove shall keep adjacent pipes in alignment. The seal shall be provided by an external rubber gasket spanning the joint and held in position by two corrosion resistant metal bands on either side of the joint. Each band shall have provision for tightening it to the required tension by means of an adjusting nut on a threaded bolt, all such parts being of approved corrosion resistant metal.

Asbestos-Cement Pipe Rubber Ring Joint

Asbestos-cement pipe manufacturers have an excellent joint which consists of a collar and two rubber rings. The ability to machine the pipe ends of the asbestos-cement allows very accurate tolerances to be obtained. This joint is a combination of the joints just described. The round gasket is placed on the spigot and when it is pushed into the collar, the rubber ring snaps into a groove in the collar.

Other types of pipe and joints may, if approved by the Engineer, be permitted. Before accepting pipe and/or joints other than those specified, the supplier of the proposed alternate pipe shall, in the presence of the Engineer or his representative, carry out all tests herein specified and such other tests as the Engineer may require. Merely increasing the slope of the tongue and groove shall not be considered an alternate joint.

TESTING FOR LEAKAGE (FILTRATION)

When the Commission refers to the infiltration or exfiltration testing of sewers, it is understood that they are 36" in diameter or less. The specifications for joints in the manufacture of sewer pipe of larger sizes is more flexible and a slope on the bell and spigot of from 5^{o} to 8^{o} is allowed. These larger pipes can be inspected and repaired from the inside.

The limiting amount of infiltration and exfiltration allowed by the Commission has already been mentioned (infiltration - 300 I.G./"Ø/m/d and exfiltration - 400 I.G./"Ø/m/d) but this does not mean very much unless we state the conditions of the test.

There are two main divisions of pipe testing and they are:

1. in the manufacturer's plant, and

2. in the field.

TESTS AT MANUFACTURERS PLANT
(As outlined in OWRC Specifications)

For the details on the testing of materials, steel, batching, mixing, curing and finishing, I refer you to the OWRC "Manufacturer's Prequalification Requirements".

Since this paper deals with infiltration, I will only describe the tests made to determine the adequacy of the rubber joint in the plant.

Pipes in Alignment

Three pipes are oriented in a machine, bulkheads are placed in the ends of the pipe and these bulkheads are restrained longitudinally, except that the joints shall be allowed to open 1/4" per joint. The pipes are filled with water, taking care to remove all entrapped air, then subjected to a hydrostatic pressure of 10 lbs. per sq. inch. At this pressure there should be no leakage.

Pipes Deflected Horizontally

Upon completion of the test in proper alignment, the test section is then deflected horizontally until at least two of the joints have been deflected to the maximum amounts shown in the manufacturer's standard installation instructions. The test section is then subjected to 5 p.s.i. internal hydrostatic pressure and at this pressure the joints should show no leakage.

Differential Load Test

The three pipes in the test section are supported on blocks or otherwise so that the two end pipes are firmly supported and the middle pipe is freely suspended, bearing only on the rubber gaskets at the joints. The suspended pipe is then loaded in accordance with the table of values which depends on the diameter of the pipe and its length. While under this load the joints should show no leakage with an internal hydrostatic pressure of 5 p.s.i. The blocking under the supporting pipes shall be arranged so as to distribute the load transferred to these pipes in an adequate manner.

The test loads for pipes under differential load are as follows:

PIPE SIZE	LOAD PER FOOT LAYING LENGTH UP TO 4'	TOTAL LOAD FOR PIPES 4' & OVER
6"	1,000	4,000
8"	1,300	5,200
10"	1,400	5,600
12"	1,500	6,000
15"	1,850	7,400
18"	2,200	8,800
21"	2,500	10,000
24" and over.	2,750	11,000

THREE EDGE BEARING TEST

In order to check the specified structural strength of the sewer pipe, the three edge bearing test is used in accordance with A.S.T.M. C-76 or C-14. The pipe is placed in a testing machine in the same fashion as it would be when installed in the trench. With the bearing on the pipe in three locations, two on the bottom and one on the top, the pipe is loaded until a 12" long, .01" wide crack is produced. This is considered the maximum design load in reinforced concrete pipe and failure in non-reinforced pipe. The reinforced pipe is then taken to ultimate failure which is usually 30% higher than that to produce the .01" cracks.

FIELD TESTS

Although the test requirements have been obtained in the manufacturer's plant, field testing is required and there is a great deal of difference between a clean production area and working in a pipe trench.

The skill of the pipelayer also has a great influence on how correctly the pipe is installed.

The field tests which can be performed on a sewer line are as follows:

1) Observation or sight test.

2) Alignment test.

3) Infiltration or exfiltration test.

4) T.V. or photographic camera inspection.

Again remember that we are speaking of pipe of 36" diameter and under.

Sight Observation Test

This is performed by sighting through the sewer main with a light placed in a manhole at the end of the test section and viewing this light from the adjacent manhole. This test is successful in short sections where the sewer is not laid in an arc.

Alignment Test

This test is performed by pulling a ball of wood or hard rubber, which is 2" less in diameter than that of the pipe, from manhole to manhole. If this ball is successfully pulled through the sewer, it is felt that the pipes have been aligned properly.

Infiltration or Exfiltration Test

The selection of the test depends on the presence or lack of ground water. If the existing ground water is a minimum of 2 ft. above the crown of the pipe, an infiltration test is performed. If the ground water is less than 2 ft. above the crown of the pipe, an exfiltration test is performed.

The infiltration test consists of isolating the test section of sewer with bulkheads and, at the downstream manhole, measuring the amount of water entering the test section over a period of time. The amount of infiltration allowed by the Commission is 300 I.G./"Ø/m/d.

The exfiltration test consists of isolating a section, including the manholes, at either end. This section is filled with water, allowing the entrapped air to escape, and left for approximately 24 hours so that the pipe walls will be saturated. The head on the pipe is obtained by filling the section until the water is 4 ft. above the crown of the pipe in the highest manhole, providing the head over the crown of the pipe at the lowest manhole does not exceed 15 ft. This test includes the leakage in the manholes. This would mean that the minimum pressure would be approximately 2 p.s.i. and the maximum 6 p.s.i. The test is accomplished by measuring the amount of water lost in the upper manhole during a time interval. The allowable exfiltration, as specified by the OWRC, is 400 I.G./"Ø/m/d.

All lateral branches included in the test section shall be taken into account in computing allowable leakage. Also, an allowance of 0.2 gallons per hour per foot of head above the invert shall be made for each manhole included in the test section. If the test produces more than the allowable leakage, the contractor shall test manholes separately.

T.V. Inspection

Regardless of the precautions taken, faults do occur. The minor faults in pipes of 30" dia. and over can be repaired from the inside. In smaller diameter sewers even minor faults could be a source of serious trouble later on. In order to locate faults in the pipe which have been disclosed by field tests, a closed circuit television is used by many contractors and owners.

A T.V. inspection consists of pulling a T.V. camera through the sewer main and observing the picture transferred to the surface on a television screen. The cable attached to the camera is graduated in feet so that the exact distance from a manhole to the fault can be determined.

In OWRC Specifications there is a section allowing the Engineer to use this method of inspection when he has any suspicions concerning a sewer installation. The OWRC Specifications stated that "if faults are found the

contractor pays not only for the repairs but for the cost of the T.V. inspection. If no faults are found the cost of the T.V. inspection is paid for by the Commission".

We have had good results with this system of inspection. Most of the difficulty is the interpretation of the picture on the television receiver. For a permanent record photographs are taken of the T.V. screen.

Another method of inspection of small diameter sewers used by the OWRC is that by a camera. This consists of pulling a camera through the pipe and taking pictures at 2' to 4' intervals. The location at which these pictures are taken is recorded in a similar fashion to that of the T.V. camera.

Problems arise when:

1. Sewers are partly filled with water or dirt. This tends to either cover or dirty the lens resulting in poor pictures.

2. Cold weather is always a problem, the lens tends to fog.

CONCLUSIONS

In summing up, the points to be remembered in order to reduce infiltration to a minimum in sanitary sewage collection systems are:

1. The class of pipe must be properly selected to suit the installation.

2. The pipe itself must be manufactured to close tolerances both in the material used and in the pipe dimensions.

3. Adequate field supervision and inspection should be maintained at all times.

4. There must be adequate testing in the plant and in the field.

RECOGNIZING AND MINIMIZING INDUSTRIAL WASTE PROBLEMS

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Of ten thousand industries in Ontario, about 20% or 2,000 use water in their operations. Of these two thousand, about one-third discharge their process wastewater directly, either with or without treatment, into lakes, rivers or other streams. The remaining two-thirds or 1,300-1,400 industries discharge their wastes into the local sewerage systems. These industrial wastes are very different from each other. Some are organic, some inorganic, some are acid, some alkaline, some contain suspended matter, some do not. Not only are the wastes from each industry different from each other but the waste from any one plant may vary widely in quality and quantity from day to day and even from hour to hour. Therefore, in the design and operation of a sewage treatment plant, it is important to know the characteristics of the industrial wastes to be included in the sanitary sewerage system.

SEWER-USE BY-LAW

In the Basic Course, the principal features of a municipal sanitary sewer-use by-law were outlined. In this by-law, there is normally a section allowing a municipality to accept industrial wastes not complying with one or more of the by-law limits but still amenable to treatment. In either case, keeping a close watch on the industrial effluents to a sewerage system is required to minimize the danger of a sewage plant or the sewers from becoming overloaded. Municipalities are advised to carry out regular sampling of industrial flows to the sanitary sewers for this purpose. Industrial wastes maintained within the by-law limits or within the limits of any special agreement between the municipality and an industry are capable of being treated. It is the responsibility of the industry to ensure that its wastes conform to these specifications. The third lecture on industrial wastes to be given at the Senior Course will

outline the pretreatment steps that can be taken at the industry to reduce the waste load to the sewers to meet the requirements of the municipality.

ABNORMAL STP INFLUENTS

However, it is of small comfort to the operator to know that a sewer-use by-law is in effect when he gets a slug of oil from any one of a number of possible sources. As probably most of you know by experience, unexpected abnormal raw sewage flows can and often do occur. They can be the result of purely accidental spills or leaks or can be the result of negligence or thoughtlessness at the source. Whatever the reason, it is up to the sewage plant personnel to handle and treat the waste to their best ability. Turning the by-pass valve is a very poor solution.

STEPS TO BE TAKEN BY STP PERSONNEL

The first duty of the operator is recognizing an abnormal raw sewage which could upset the plant when it occurs or even anticipating an abnormal discharge before it occurs. The second step is determining the nature of the abnormal flow and its possible effects on the various units of the plant and the third and most important step, of course, is attempting to minimize the effect of the industrial waste on the sewage plant and so alleviate the seriousness of the upset which may occur.

RECOGNIZING A PROBLEM

An elaborate array of instruments for the detection of problem situations, while sometimes useful and always impressive, is not essential. We are all equipped with very sensitive and sophisticated analytical apparatus, our senses of sight and smell. A good point to remember is what has once been said and often repeated: do not observe a situation with your eyes but rather with your brain.

The means of recognizing problem situations are:

1. Flow - Excessive flow or widely fluctuating flow

can overload the sewers and your plant. If it is not raining, it can be a sign, especially at smaller plants, that an abnormal waste is being received. High flows can easily be detected by noting the water height in channels.

2. <u>Odour</u> - When the characteristic odour of raw sewage or the mixed liquor changes, trouble can be expected. Septic raw sewage can be detected in this way. Many organic chemicals have characteristic odours by which they can be detected and although they may not be identified may give you a clue as to the source of the waste. Sulphides have the characteristic "rotten egg" odour; cyanide has a burnt almond odour; phenol has a characteristic antiseptic odour.

3. <u>pH</u> - The pH can be determined very easily by using wide-scale pH paper. A pH meter is not necessary. You must take care to seal your pH paper in an air-tight container when not in use as fumes in the atmosphere can degrade or discolour the paper. This is especially important in industrial areas where gas from stacks may be emitting acid materials to the atmosphere. The pH paper will tell you roughly how acid or alkaline the sewage is.

4. <u>Temperature</u> - Temperature is another important parameter for indicating abnormal wastes, again particularly at smaller plants. Wastes which can cause a temperature increase are hot alkaline or detergent cleaning solutions used in the metal-finishing industry, metal pickling solutions which also have a very low pH and high iron content or cooling water which should not normally be allowed to enter the sanitary sewers as it is usually relatively clean. Warm sewage can become septic before reaching the sewage plant.

5. <u>Colour</u> - Colour changes in the raw sewage can easily be detected. Industries which discharge coloured wastewater include textile mills (dyes), slaughterhouses (blood), milk processing plants (milk wastes) and metal-working industries. Some chemical solutions are highly coloured as, for example,

copper	-	blue to green
iron	-	reddish brown to yellow
chromium	-	green or yellow

nickel - green
pickling liquors - green

6. Suspended or Floating Solids - The presence of telltale fibres from a textile plant, inorganic solids such as clays or sand from a foundry, feathers, etc. will often enable the operator to determine the cause of a problem. Sludge picked up on the bar screens or found at the pumping stations can also provide information as to the source and nature of the objectionable waste.

7. Oils and Greases - These are, of course, readily apparent to the operator.

8. Examination of Sewers and Pumping Stations - An examination of the sewers and pumping stations can help to pinpoint the source of wastes not meeting the by-law limits. Items to check include:

(a) corroded sewers or concrete work;
(b) corroded or eroded pump impellers at the pumping stations;
(c) fibres in manholes or pumping stations;
(d) excessive sediment in pumping wells;
(e) grease trapped in stagnant areas.

KNOWLEDGE OF INDUSTRY

A thorough knowledge on the part of the sewage plant personnel of the industries serviced by a sewage plant is essential in diagnosing the cause of a problem. In larger municipalities, this is more difficult than in smaller municipalities where there may be only one or two major industries. However, the operator should know the characteristics of the wastes discharged by each industry and also the nature of any possible batch or seasonal discharges. This knowledge can be useful in anticipating increased loadings to the plant. There is included in these notes a list of some of the various sources and characteristics of industrial wastes.

Greatly increased seasonal loadings can be expected from the fruit and vegetable processing industry. Batch discharges at many types of industry can occur at any time but usually follow a pattern and are especially common at the end of an operating day or week when spent solutions are sewered or when equipment or plant wash-ups are performed.

Before designing a sewage treatment plant, an industrial waste survey of the municipality is required to determine the total industrial waste load. The quantity, concentration and characteristics of each individual major waste stream is determined in such a survey. Data which should be quickly available to an operator would follow the same pattern and would include:

(a) name of industry;
(b) location of industry (point of discharge and pumping station or trunk sewer serving the industry);
(c) type of industry;
(d) raw materials and products;
(e) operating schedule;
(f) week-end or night operations;
(g) volume of wastewater sewered (average and maximum);
(h) normal characteristics of wastewater;
(i) pretreatment facilities (failure or by-passing of pretreatment facilities may cause problems at the sewage plant);
(j) nature of any possible batch discharge.

RECORD OBSERVATIONS

Well kept records of conditions at the sewage plant during normal operating conditions as well as a complete set of observations during upsets will assist in determining the source of an objectionable industrial waste if this cannot be immediately determined. Perhaps of more importance, these written records invariably have a much greater effect than mere verbal reports on the industry responsible and they also provide the municipal officials with data to show that

improvements are required at the industry. Items to record are:

(a) time and date of abnormal waste flow;
(b) duration of abnormal flow;
(c) conditions observed as noted previously.

Many times these objectionable intermittent industrial waste discharges are the result of sloppy housekeeping at the industrial plant, thoughtless action, or faulty equipment. Much can usually be done at the source to prevent a re-occurrence and therefore it may be a good idea not only to inform the industry of the effects of its waste but also to invite officials of the industry to visit the plant when an upset is occurring to impress upon them the effects of their negligence. Many people give no further thought to their wastewater once they have gotten rid of it and such a visit will make them aware that there are people who must handle and treat the waste before final disposal.

SAMPLING

In addition to recording all observations, samples of the raw sewage and from the sewer area suspected should also be taken for later analysis. While these are not immediately useful, they may become essential in ultimately locating the source. The analytical results are also extremely important when the municipal officials approach the industry to enforce the by-law. The analyses required will depend on the specific conditions. Samples of sludge found on bar screens, grit chambers, in manholes or pumping stations should also be collected if it is felt these could be useful.

TEMPORARY SOLUTIONS

Permanent pretreatment at the source will be discussed in the Senior Course. The temporary short-term solutions at a sewage plant to problem situations usually require much more ingenuity and quicker action on the part of the operator than is required in designing permanent solutions. A knowledge of the capabilities and the flexibility of your plant is also essential.

Treatment is deteriorating and you are required to take action. What do you do?

1. Septic Sewage - will cause septicity in primary clarifiers and increase the oxygen demand in the aeration section. Pre-aeration or prechlorination of the sewage may help.

2. Frothing - attempts to control frothing can be made by maintaining a high solids level, using a water spray or using defoamers such as tall oil.

3. High Organic Loading - Generally a higher solids concentration and increased aeration can be used. Sometimes changing plant operation to contact stabilization is done.

4. High Suspended Solids - If organic and if not removed in the primary clarifier, generally a higher solids concentration and increased aeration, as above, can be used. Alkaline reagents may have to be added to the digestor to maintain a proper pH due to the increase in volatile acid production if this occurs frequently. Additional sludge hauling may also be required. Polymers or coagulant aids such as alum or ferric sulphate can be added in the primary clarifiers to assist in solids settling.

5. Large Volume-Low Strength Sewage - During rainstorms when a diluted sewage may be received, problems may arise because of a lack of sufficient aeration solids. Ferric chloride can be added to assist in coagulation of solids.

6. Toxic Materials - In this group are the heavy metals: nickel, copper, chlomium, etc. and cyanides and phenols. Higher solids in the aeration chamber may help to minimize their effects. These materials should be suspected when upsets in either the aerobic or anaerobic sections occur for no apparent reason. Also if storm water storage tanks are available, the sewage can be stored and later bled in at a controlled rate. The same holds true for high strength organic wastes.

7. Oils and Greases - These can be skimmed off manually.

8. Odour Control - Unusual odours can be generated for a number of reasons during upsets. Chlorination can be used to reduce the intensity of some odours. Commercial products are also available to mask odours. A plant operating normally will not give rise to objectionable odours and therefore chemical addition should not take the place of good housekeeping and plant cleanliness.

9. Control at Source - If the source is known at the time, the industry should be informed and requested to check its operations and to stop the objectionable discharge.

Characteristics of Some Typical Industrial Wastes

Industry	Origin of Major Wastes	Major Characteristics
Canning	trimming, culling, juicing, blanching of fruits and vegetables. Plant washups	suspended solids, colloidal and dissolved organic matter, colour.
Dairy Products	waste buttermilk and whey, dilutions of whole and separated milk, plant washups.	dissolved organic material, milky appearance.
Brewing and Distilling	steeping and pressing of grain, residue from distillation of alcohol, wash water	dissolved organic solids containing nitrogen, odour characteristic of yeast.
Meat-packing	slaughtering of animals, rendering of bones and fats, residues in condensates, wash water	may be coloured, contain grease and solids such as feathers.
Soft Drinks	bottle washing, plant washup	high pH, suspended solids, BOD.
Textiles	rinse waters, dye solutions, detergent cleaning solutions	highly alkaline, coloured, high temperature, fibres.
Tanning	unhairing, soaking, deliming, bating of hides	high total solids, hardness, sulfides, chromium, pH, precipitated lime and BOD.

Industry	Origin of Major Wastes	Major Characteristics
Laundries	washing of fabrics	high turbidity, alkalinity, organic solids
Detergents	washing soaps and detergents	high in BOD, phosphates, floating fatty acids, foam.
Insecticides	washing of products	toxic organic compounds.
Metal Plating	cleaning of metal, rinsing of plated objects.	acids, metals, cyanide, alkaline.
Iron-Foundries	wasting of used sand	high suspended solids, mainly sand.
Glass	polishing and cleaning of glass	red colour, alkaline, non-settleable solids.

DESIGN OF A SANITARY SEWAGE COLLECTION SYSTEM

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A. INTRODUCTION

The following paper is intended to give the Sewage Works Operator a general appreciation of the basic concepts in the design of sewage collection systems. Although the operator's job is more closely tied with the actual operation of the system, it is beneficial to have a general knowledge of how and why sewers, pumping stations and appurtenances are selected. This paper deals primarily with the planning and design of the system.

B. GENERAL ASPECTS

Whether designing a small section of sewer to serve a newly developed portion of a municipality or an entire collection system, there are certain overall considerations which must be examined. The designer must forecast what the future of the particular area will be. The forecast will be more accurate if an official plan exists upon which the designer can base future land use predictions. Some of the factors affecting the design are listed below:

1. Area to be serviced

The area under study may be limited by topography, political boundaries or by economic factors. When providing sewers and pumping stations, etc. for an area one should look beyond the boundaries set down for the immediate study and make provisions, if necessary, for the inclusion of adjacent areas at a later date.

If an official plan is available, then the selection of the areas within the municipality have already been outlined and the need for sewers and other services have had some consideration. Land use patterns are predictable and scattered development may be avoided by proper zoning by-laws.

2. Land use

The actual nature of the development in the area may be dictated by political and/or economic considerations; however, in modern urban areas planning has helped reduce scattered and chaotic growth. All municipalities contain residential, commercial and usually industrial areas. One of the problems in the design of the system is to protect the location, nature and extent of each of these areas.

It is obvious that the size of a sewer required will be quite different on a suburban residential street than on a road containing high-rise apartment buildings. Industrial sewage flow may vary over a wide range from a large effluent producing industry, (possibly more than the total domestic contribution), to a dry type industry in which only domestic sewage from the plant personnel would be discharged to the sewer system.

The land use pattern must be carefully studied to give the designer some idea of the quantity and nature of sewage that will be discharged now, in the future, in a local area, and in the entire municipality.

3. Population prediction

Relative growth of an area or municipality may be dictated by many factors such as:

(a) proximity to large urban areas

(b) number and type of industries present

(c) government policy

(d) economic situation of the region

The effect of Metropolitan Toronto on the surrounding municipalities need not be emphasized. The number and type of industries present will affect the population growth of a town, in that the industry already established will have necessitated a certain type of service being made available to the industry, thus perhaps allowing additional industry to be served at little inconvenience. The Provincial Government may have declared a certain area to be "depressed". In such an area tax relief may be given to industries which will settle there, thus making this area more attractive than another area, all other things being equal. The municipality may be very aggressive in advertising for new industries and convincing representatives of industry to locate in their municipality. The economic prosperity of a region will very much influence the availability of that area to progress.

All of the above factors will influence the rate at which population increases and hence the need for municipal services such as sewers.

C. DETAILED ANALYSIS OF THE AREA

After an analysis of the general aspects of design, it becomes necessary to closely examine the system and the various factors influencing design.

1. Analysis of the area

The topography of the area - soil conditions, natural boundaries, and ground elevations will determine the physical layout of the sewage collection system. It is necessary to select the natural drainage areas and the routes of the collectors, sub-trunks and trunk sewers.

2. Soil conditions

Knowledge of soil conditions of the area is essential. Soil conditions may render the provision of sewer services in some areas uneconomical. Poor

soil conditions will affect the cost estimates as well as the construction methods and materials. Should a large quantity of rock be necessarily excavated then obviously the cost of the system will be great as compared to that for sand soil conditions. The conditions such as rock or muskeg may require that water and sewer services be provided in a common trench to prevent excessive excavation and/or fill material.

Soil conditions may also affect the type of materials used. In soil conditions where good bedding cannot be easily obtained, or where the soil is corrosive, then plastic forcemains may be more economical and suitable than asbestos cement.

3. Population prediction methods

Before attempting to calculate the anticipated flow, it is necessary to estimate the future population of the area. The interpretation of past records may be used in conjunction with factors mentioned previously under "general aspects, sub-section 3". Several methods may be used to predict future populations from past records, including:

(a) graphical extension of past records into the future

(b) comparison with similar cities of somewhat greater population

(c) mathematical analysis using arithmetic progressions, geometric progressions, logistic S curve or ratios.

Method (a) above is one of the simplest and most frequently used methods of predicting future population.

At this point, it is important to make the distinction between projected population and serviced population. It is probable that all the population cannot be serviced for economic or physical reasons. For example, it may be uneconomical to service two houses at a low elevation either by a sewage pumping

station or by lowering a sewer. Thus providing the septic tanks are working satisfactorily, it may be worthwhile to not service these houses. The factor stressed is that only the serviced population will contribute to sewage flows.

4. Distribution of population

In order to effectively design a system, it is necessary to know not only the future population of the community, but also the densities throughout the areas serviced by the sewage collection system. The average density may be obtained by taking the total predicted population and dividing this number by the habitable area of the community.

It is essential for a modern community to have an official plan of development. Knowing the nature of the development to take place in various sections of the community, the designer can assign probable saturation densities as determined from previous experience. In this manner, an estimate can be made of the population of tributary areas for each sewer involved. The saturation density of all sectors of the community will undoubtedly exceed the predicted design population of the community as it does not usually happen that the community is fully saturated with people at the design period.

5. Flow prediction

(a) Sewage Flows

With a reasonable knowledge of soil conditions, topography, area, land use and populations, the problem of estimating sewage flows may be attempted. Waste flows from the following sources may be summed up to give the total flow:

(i) domestic

(ii) industrial

(iii) commercial

(iv) institutional

(v) infiltration of groundwater

(vi) storm water

Values of the first four (4) may be obtained by analysis of the problem and application of information gained through past experience. The effect of infiltration is minimized by good construction; the effect of storm water is minimized by eliminating combined sewers, roof drains and footing drains where possible.

(b) Common Expedient

Due to the obvious difficulty in obtaining predictions, the designer frequently finds it a satisfactory expedient to simply assign a per capita flow which is meant to include all contributions. Such a value may be obtained by examining past water consumption records and per capita sewage flows from similar municipalities. A common figure used in design for average day sewage flows is 100 gallons per capita per day.

(c) Actual Design of Sewers and Appurtenances

With the information now available, the designer can proceed to select the actual sewer sizes. Items which must be considered now are design period, design flow and hydraulic factors. See Appendix A for a list of short forms used in design.

(i) Design Period - For lateral sewers it is conservative to design for the saturation tributary population and to assume that the period of design will encompass total land use. In the case of trunk sewers, interceptors, major relief sewers and outfall lines, it becomes uneconomical to design for the saturation population. However, it would also be uneconomical to use a short 10 or 20 year design period, and a design period of 40 - 50 years, compatible with the life of the sewer, is considered good practice.

(ii) Design Flows - The flows computed for the tributary areas will be the anticipated average day flows. The designer must select his sewers to carry the maximum anticipated flows. For laterals and small sewers, the peaking factors are assumed to be in the ratio of 4:1, whereas factors of 2.5:1 or 3:1 are used in the design of sewers with larger tributary populations. The following formula may be used to determine the peaking factor:

$$\text{Peaking Factor} = \frac{\text{maximum expected flow}}{\text{average daily flow}}$$

$$= \frac{5}{P^{0.2}}$$

$$\text{or} \quad 1 + \frac{14}{4 + P^{\frac{1}{2}}}$$

where P is the tributary population in 1,000's.

No peaking factor is normally applied to the infiltration flows; reduced peaking factors are employed for industrial contributions, since industrial peaks can be normally predicted with greater certainty than can domestic peaks.

(iii) Grade - The grade of the sewers will be influenced considerably by the ground surface. That is, in order to provide a fairly uniform depth of sewer, the slope of the sewer must generally follow the existing slope of the ground. Sewer grades must provide self-scouring velocity at full flow conditions; that is, the minimum velocity at full flow conditions must be at least equal to two feet per second, the velocity at which settling out of solids will be prevented. A partial list of minimum grades is presented below:

DIAMETER OF SEWER (inches)	MINIMUM GRADE (ft./1000 ft.) @ n = 0.013
8	4.0
10	2.8
12	2.2
15	1.6

DIAMETER OF SEWER (inches)	MINIMUM GRADE (ft./1000 ft.) @ n = 0.013
18	1.2
21	1.0
24	0.8
27	0.67
30	0.58
36	0.46

In general, the cost of sewer construction will increase in proportion to the depth of the trench, modified by the condition of the soil, depth of rock, location of groundwater, need for shoring and other sub-surface factors. Sewer depths greater than 15 feet are considered inadvisable and a minimum depth of cover of six feet is usually acceptable, but the sewer must be deep enough to permit basement drainage. In some cases, it may be advisable to provide a few houses with sump pumps to pump the basement drainage up to the sewer rather than lowering the whole sewer significantly and thus greatly increasing the cost of the sewer at a lower depth. There have been specialized cases where a complete system of sewers have been installed shallowly to keep the cost of the sewers from being prohibitive with sump pumps in almost every house to provide basement drainage.

(iv) Size of Sewer - After the designer has determined the sewage flows and grade limitations, he may proceed to select the appropriate sewer size. Formulaes relating capacity, and hydraulic gradient, co-efficient of friction, and pipe sizes are available. One of the most commonly used of these formulaes is the Manning formula:

$$V = \frac{1.486}{n} R^{2/3} S^{\frac{1}{2}}$$

where V = the velocity of flow in feet per second
R = the hydraulic radius (½ radius for circular pipes flowing full)
S = hydraulic gradient = the slope of the pipe in feet/foot assuming uniform flow

n = co-efficient of roughness (varies with type of pipe)

Nomographs for the solution of this equation have been developed for easy use. Such a nomograph is attached as Appendix B at the end of this paper.

To improve maintenance and prevent blockages, a minimum sewer size of eight inches is required in the Province of Ontario. In the average community, most laterals need only be of the minimum size.

(d) Pumping Stations

(i) Requirements for a Pumping Station

In general, sewage pumping stations may be required under the following conditions:

1) Where topography is such that good sewer grades by gravity flow are not possible without excessive depth and with consequent high construction costs. The sewer grade is carried as low as is practical, and then the sewage is lifted by a sewage pumping station in order that the downstream sewer may be placed closer to the ground surface, thus reducing construction costs. This case is encountered where the topography is very flat.

2) Where the hydraulic gradient is such that there is insufficient head for gravity flow through a treatment plant.

3) Where it is necessary to boost the sewage over a ridge through a forcemain, to a point from which it will flow by gravity.

(ii) Types of Sewage Pumping Stations

The designer may choose a wet well/dry well type of pumping station or one utilizing submersible pumps.

The first type consists of a wet well into which the sewage flows and a dry well in which the pumps and motors are located; the pump suction line

extends from the dry well into the wet well.

The station utilizing submersible pumps contains a wet well only, with the pumps submerged in the raw sewage. The submersible type pumps are usually limited to approximately nine horsepower and thus this type of station would be considered a small station.

Wet wells are generally built-in-place, whereas the dry wells may be either built-in-place (custom-built) or prefabricated of steel and installed on the site. The wet well/dry well type of sewage pumping station may have either an above ground motor room or a prefabricated package type dry well in which the dry well is completely underground. In the package type station where the pumps and motors are below ground, the horsepower is usually limited to 50 due to space, maintenance and ventilation factors. These stations would be considered a medium-sized sewage pumping station. The above ground custom-built type sewage pumping station may be used either as a medium-sized pumping station or a large pumping station. It is noted that the dry well/wet well type of sewage pumping station can be expanded more easily than the pumping stations using submersible pumps.

(iii) <u>Pumping Station Capacity and Pump Selection</u>

The pumps are normally sized either for 10 or 20 year maximum expected flow. If the difference between the two is not great then 20-year maximum flow is generally used for the design. It is common practice to provide 100 per cent standby for the largest pump, resulting in two equal pumps for small stations where each pump is capable of pumping the maximum expected flow. Peaking factors are computed in the same manner as for the sewers as outlined previously in Section (c), Sub-section (ii).

The wet well is sized for the 20-year flow such that a maximum of six motor starts per hour will occur at the critical condition. In a two-pump station where each pump is capable of pumping the maximum expected flow, the critical condition is when the influent in gallons per minute is half the pump capacity in gallons per minute. Wet wells are benched to prevent deposition of solids in

dead areas such as corners. Screening is provided, where possible to protect the pumps from rags, wood pieces, etc.

It is good practice to equip the pumping station with an overflow in case of forcemain, mechanical, or electrical failure. Standby power may be provided to prevent overflows into waters where an occasional overflow of raw sewage cannot be tolerated, such as upstream of water works intakes, recreational streams or backup into basements. Standby power must be provided if for topographical reasons an overflow cannot be provided and danger of backup into basements causing a public health hazard, is possible. Standby power may be provided in the form of either a diesel generator set or a direct-drive diesel engine. If standby power is provided at a sewage pumping station, it must be located in an above-ground structure.

(iv) Forcemain

The forcemain from the station must be selected to minimize friction losses but at the same time to provide a minimum scouring velocity of 2.5 feet per second in the pipe to prevent deposition of solids. The maximum velocity in a forcemain is usually limited to a maximum of ten feet per second since velocities in excess of this figure will cause very severe head losses and resulting high heads required for the sewage pumps and high power costs.

(e) Inverted Syphon

The purpose of the inverted syphon or depressed sewer is to carry the flow under an obstruction such as a river or depressed highway and to gain as much elevation as possible after the obstruction is passed.

An inverted syphon is an arrangement whereby the sewage flows down the upstream side of the syphon and is forced up the downstream side by the pressure of the in-flowing sewage. Syphons generally consist of two or more pipes (barrels) with the smallest laid in the lowest position and designed to take minimum flows;

higher flows spill into the second pipe during peak flow conditions. Velocities in the range of three feet per second should be achieved to prevent blocking of the syphon. Control gates must be provided at both ends so that either pipe may be removed from service. Drains and clean-outs should also be provided to allow maintenance if necessary.

(f) Manholes

The purpose of manholes is to permit inspection, cleaning of sewers, removal of obstacles and sometimes flow measurement. Manholes are located at the junctions of sewers and at the changes in alignment or grade and diameter. Street intersections are common locations for manholes, as junctions frequently occur there. It is common practice to have a manhole at the upstream end of a sewer for convenience in flushing and cleaning, and in the event of a future extension of the sewer. The recommended distances between manholes on a straight run are as follows:

- 400 feet for sewers 15-inch diameter or less
- 500 feet for sewers 18-inch - 30-inch diameter
- for sewers greater than 30-inch diameter the spacing may be increased especially where sewers are big enough where a man can actually walk through. The spacing however, also depends on the cleaning equipment which the municipality has. If modern and efficient cleaning equipment is present in the municipality then the manhole spacing may be greater than a similar municipality which might have poor cleaning equipment.

Manhole spacing is affected by grades available. If flat grades are necessary then manholes should not be spaced too far apart since blockages are more likely than if good sewer grades are possible. At the other extreme, sewers going down steep hills may require "drop" manholes, closely spaced, to dissipate energy.

(g) Example

Given: A village is situated as shown in Figure No. 1.

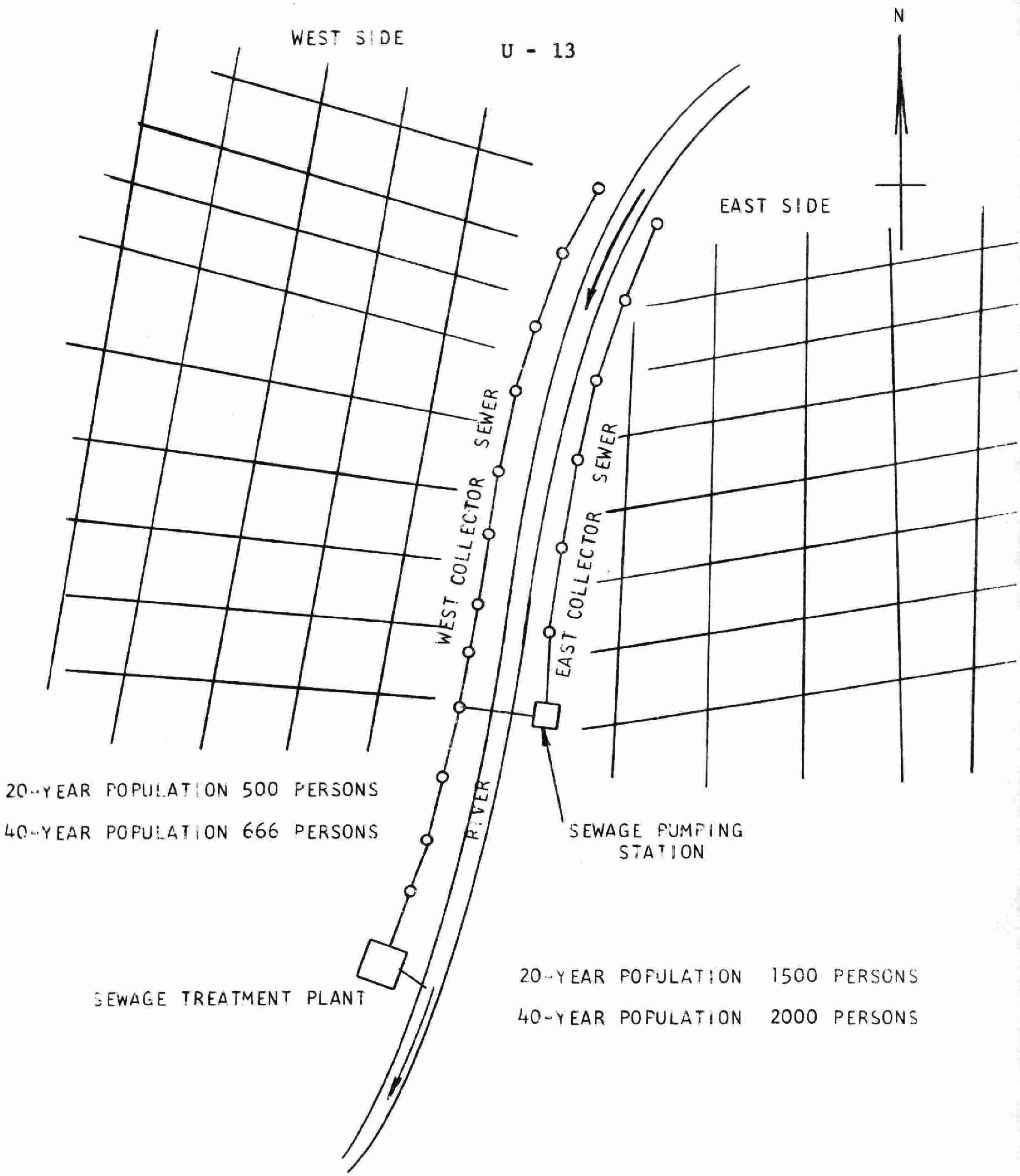

WEST SIDE
N
EAST SIDE
WEST COLLECTOR SEWER
EAST COLLECTOR SEWER
RIVER
20-YEAR POPULATION 500 PERSONS
40-YEAR POPULATION 666 PERSONS
SEWAGE PUMPING STATION
SEWAGE TREATMENT PLANT
20-YEAR POPULATION 1500 PERSONS
40-YEAR POPULATION 2000 PERSONS

FIGURE NO. 1

After analysing future trends it is found that this small community is going to develop in 20 years with a mixture of residential, commercial and small industrial areas to approximately 1,500 persons on the east side of the river and 500 persons on the west side. The ideal site for the sewage treatment plant is shown south of the village on the west side of the river.

The designer has examined past records and determined that the average water consumption has been 65 gallons per capita per day. He has further decided to use a 20-year design period for his treatment plant. Since the community is small and of balanced composition, he has determined that he can conservatively use a per capita sewage contribution of 80 gallons per capita per day - this will account for the current trend toward increased water usage.

An analysis of the topography shows that the village can be separated into two drainage areas with collector sewers on each side of the river. A sewage pumping station will be required at the east side of the river; it will discharge into a manhole on the west side; from there the sewage will flow by gravity to the plant.

An examination of the drainage area shows that all laterals need be only eight-inch diameter - even for the 50-year life of the sewer. The design of the collector sewers may be computed as follows:

i) East Collector

Basic design data - east side	-	1,500 persons - 20-year population
east side	-	2,000 persons - 50-year population (saturation)
Average per capita contribution	-	80 gpcd
Maximum expected flow	-	2000 x 80 x peaking factor

$$\text{Peaking factor} = 1 + \frac{14}{4 + P^{\frac{1}{2}}}$$

$$= 1 + \frac{14}{4 + 2^{\frac{1}{2}}}$$

$$= 3.56$$

Therefore: Maximum expected flow = 2000 x 80 x 3.6

= 575,000 I gpd

Infiltration = 35,000 I gpd

610,000 I gpd

= 732,000 US gpd

Sewer size - selected from nomograph (appendix B) at a slope of 2.4 feet per 1000 feet and with n = 0.013.

It is found that a 10.3-inch diameter sewer is required; hence, the next standard size is chosen, which is a 12-inch diameter sewer.

ii) <u>West Collector</u>

From similar calculations the designer finds that an eight-inch diameter sewer will suffice.

It would be good practice for the municipal construction inspector to verify this size by going through the calculations similar to the above calculations for the East Collector using the 20-year population of 500 persons, a 50-year population of 666, a grade of four feet per 1000 feet, infiltration of 15,000 gpd.

iii) Pumping Station

10-year population	-	1,200 persons
20-year population	-	1,500 persons
10-year average flow	-	1200 x 80
	=	96,000 gallons + infiltration
20-year average flow	-	1500 x 80
	=	120,000 gallons per day + infiltration

Since the two flows are reasonably close, the design will be for the 20-year flow.

Maximum expected flow	-	1500 x 80 x 3.68
	=	442,000 gpd
Infiltration	=	35,000 gpd
Total	=	477,000 gpd
	=	331 gpm

In order to provide a velocity of 2.5 feet per second an eight-inch diameter forcemain is required - this will also satisfy the head conditions.

It was necessary to choose a wet well/dry well type of pumping station with prefabricated dry well, for economical reasons. Two 331 gpm pumps will be provided in order to provide a standby pump in case of a mechanical failure.

iv) Trunk Sewer

The size of the trunk sewer from the manhole (to which the pumping station discharges) to the sewage treatment plant is computed by using the 50-year flow, and a slope of 2.2 feet per 1000 feet.

Since the trunk sewer must be designed on the 50-year flow and in the absence of any other information, assume that the west side of the town grows in the same

proportion as the east side of the town. Thus, the 50-year total population of the town may be computed by the following ration:

$$\frac{\text{50-year population (east side)}}{\text{20-year population (east side)}} = \frac{X}{\text{20-year population (whole town)}}$$

where X = 50-year population of the whole town

$$X = \frac{2000 \times 2000}{1500} = 2666 \text{ persons}$$

maximum expected flow = 2666 x 80 x 3.5	=	745,000 gpd
infiltration	=	25,000 gpd
required capacity	=	770,000 gpd
	=	1.43 cfs

From the nomograph, it is found that a 12-inch diameter sewer will suffice.

D. CONCLUSIONS

An attempt has been made to illustrate some of the techniques employed in the design of a sewage collection system. Necessarily, many aspects have been simplified or ignored, since good design requires experience and patient attention to details. It is hoped, however, this outline is of some use in furthering the understanding of basic concepts behind sewage collection design.

REFERENCES

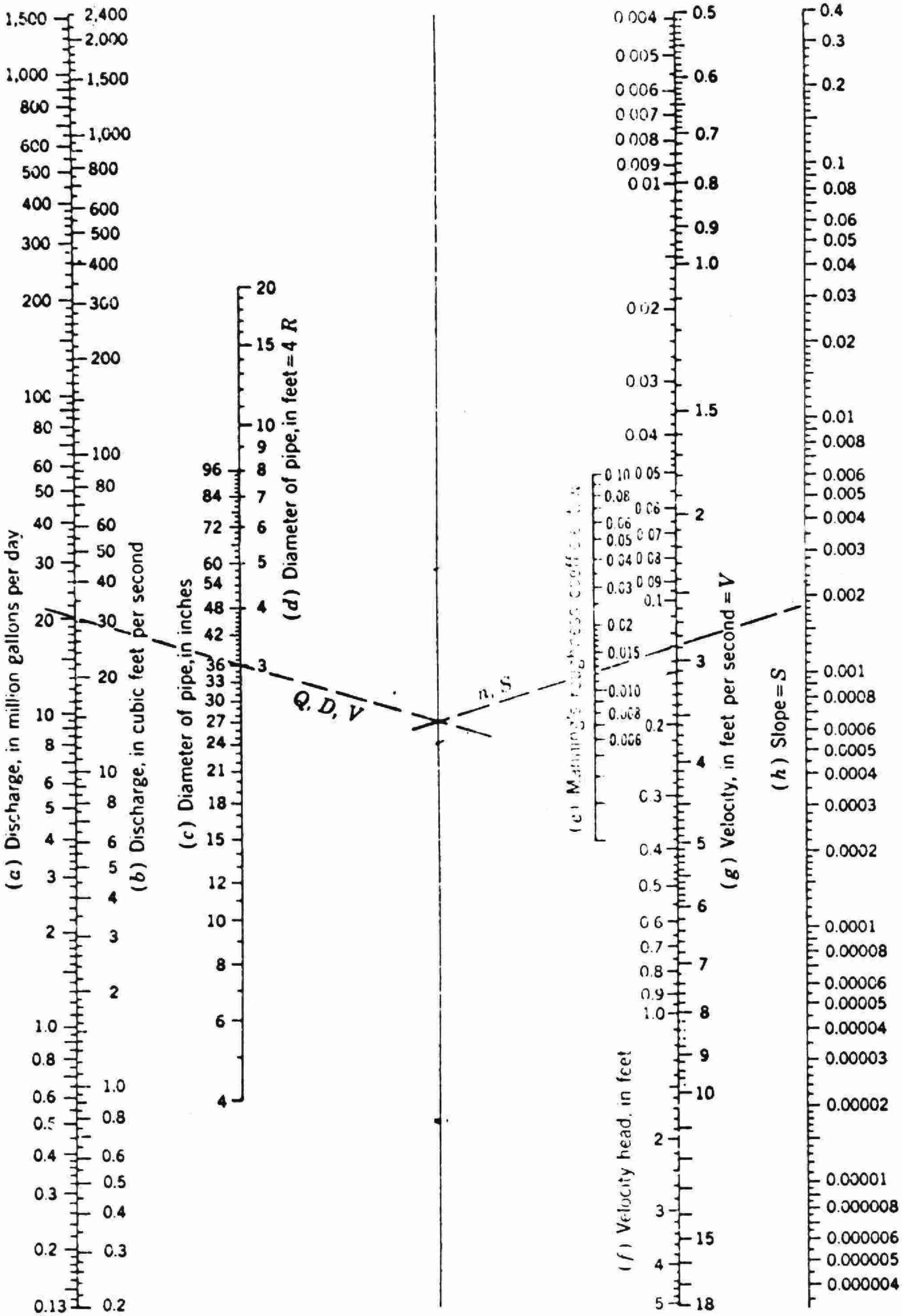
1) Great Lakes -- Upper Mississippi River Board of State Sanitary Engineers, Recommended Standards for Sewage Works 1968 Edition, Published by Health Education Service, P. O. Box 7283, Albany, N.Y. 12224

2) Joint Committee of ASCE and WPCF, Design and Construction of Sanitary and Storm Sewers, 1967.

APPENDIX A

LIST OF SYMBOLS AND ABBREVIATIONS

Q	=	discharge in cfs, mgd or gpm
V	=	velocity in fts.
A	=	cross-sectional area in ft.^{2}
H	=	head loss in feet/1,000 feet
c	=	co-efficient of friction
S	=	slope in feet/foot or feet/1,000 feet
R	=	hydraulic radius in feet
cfs	=	cubic feet per second
mgd	=	million gallons per day
gpm	=	gallons per minute
ft.	=	feet
ft.^{2}	=	square feet
fts.	=	feet per second
sec.	=	second
Ø	=	diameter in inches or feet



ALIGNMENT CHART FOR FLOW IN PIPES (MANNING'S FORMULA)

WASTE STABILIZATION PONDS

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HISTORY

The term "waste stabilization pond" is synonymous with a number of other terms such as "waste lagoon, oxidation pond, oxidation lagoon, sewage lagoon and lagoon". These ponds are shallow basins with an average operating depth of five feet. The surface area of these ponds is usually quite extensive, generally described in terms of acres.

This type of treatment is not new. In fact, it is probably one of the oldest man-made sewage devices, existing in Europe and Asia for centuries. In some parts of the world the ponds are stocked with fish for human consumption. However, this has not been practiced in North America. In North America lagoons were initially developed in Northwestern United States and Western Canada. The use of waste stabilization ponds in Ontario as a means of waste treatment is relatively new. The first lagoons were built in the Province in 1956 consisting of two four-acre cells serving the Army Camp at Ipperwash, Township of Bosanquet. Since that time there have been approximately 136 waste stabilization ponds built in Ontario to treat domestic sewage, 58 municipal and 78 private installations. In addition there are also a number of aerated lagoons treating combined domestic and industrial wastes.

TREATMENT PROCESS

The mechanism of stabilization is accomplished by biological processes. Raw sewage or pre-treated wastes entering the lagoons are stabilized by several natural self-purification "phenomena".

The purification process in the lagoon is dependent upon the combined action of wind, sunlight, temperature, sedimentation, bacteria and algae. A portion of the solids in the raw sewage settles immediately while the remainder are dispersed by wind action in the overlying water.

The bacteria, which are naturally present in the sewage and soil, feed on the organic material converting it to substances such as carbon dioxide, ammonia and nitrogen. This abundent supply of soluble nutrients provides an ideal source of food supply for the algae which grow prolifically near the surface of the pond. The algae is dependent upon sunlight for their activity and release large quantities of oxygen (photosynthesis). The oxygen in turn is used by the bacteria and is in fact absolutely essential to their activity. During periods when the sunlight is excluded the algae uses up oxygen and gives off carbon dioxide (respiration). Normally, the quantity of oxygen that the algae produce is in excess of the requirement of bacterial respiration. The air-water interface also provides oxygen to the system and the shallow design depth, long holding time and wind action all contribute to the efficiency of oxygen absorption from the atmosphere. Even in the summer there will be some anaerobic activity in the bottom layers.

Ice cover is a barrier to both light and wind. In the winter algae will continue to supply oxygen, but at a reduced rate, because of the diminution of light; and ultimately an anaerobic condition will develop. Accompanying low temperatures also slows down the bacterial action. The efficiency of treatment in terms of BOD removals may be expected to drop and odorous gases will be formed due to anaerobic decomposition. No problems develop while the ice cover remains, but odours of varying intensity and duration may occur during the spring breakup lasting a few days to a week. However, this has not been a problem in Ontario.

TREATMENT EFFICIENCY

The lack of reliable flow data presents a major difficulty in the evaluation of pond performance. An effort is, however, being made at present to provide facilities for flow measurement at all installations.

Biochemical oxygen demand (BOD) reductions of up to 90 per cent can be expected in the summer and fall. In the winter and spring BOD reduction efficiency may be reduced to 65 per cent.

Suspended solids reductions fluctuate with the seasons. Lowest concentrations are present during the months of January and February. An ice cover generally protects the ponds from wind during these two months, and algal growth and biological activity, in general, are at a minimum. During the spring breakup the contents are resuspended. By late summer the algal population develops to the extent of becoming the dominant material in suspension. An average reduction of 80 per cent can be expected.

Coliform reductions of over 99.5 to 99.9 per cent have been obtained. However, due to the high coliform concentrations (over one million) in the raw sewage the effluent still contains large numbers.

DESIGN CRITERIA AND CONSTRUCTION DETAILS

Location

The location of a waste stabilization pond should be based on several factors, namely, land costs, prevailing winds (down wind of habitation), location

of ground water aquifers, elevation, topography, soil characteristics, proximity of dwellings and proximity of receiving stream. Depending on local conditions the facility should be isolated from existing development as far as economically possible. In Ontario we have suggested 1,500 feet from the nearest house as an objective. However, as previously stated, this is sometimes not possible to obtain. Many installations within a few hundred feet of houses have not created problems. Soil characteristics should be such so as to produce a minimum percolation into the ground. A seepage rate of 1/3 inch per day may require artificial sealing of the pond bottom. Limestone formations should be avoided.

Surface Area and Shape

Organic loading to the ponds in Ontario has been limited to 20 pounds of BOD per acre per day. This is approximately equivalent to 100 persons per acre. For normal domestic wastes this loading usually provides approximately 120 days storage.

However, if yearly or half-yearly storage is required (i.e., the lagoons are sized so that there would be a discharge only once or twice a year) then the storage requirement becomes the deciding factor with respect to the acreage requirements. The present practice is to limit each cell to ten acres of surface area. Larger areas may experience dyke erosion.

The overall shape of a lagoon is not particularly important except as it relates to the surrounding topography. Round, square or rectangular shapes may be used but care should be taken to ensure that coves, islands or peninsulas are eliminated since they may interfere with circulation and develop local nuisance conditions. Where rectangular shapes are used, the length should not exceed three times the width.

Depth

The optimum operating depth varies with the season. During the winter when the ice thickness may vary from less than one foot to well over three feet, a total depth of five feet is desirable. In the early spring, immediately after the ice has been removed, a shallow depth of about 2.5 feet would encourage rapid algae growth. Provision should be made in the design of the overflow device for variable level control between 0 - 5 feet. At least three feet freeboard should be provided.

Previously lagoons were only emptied to the two-foot level for fear that aquatic growths such as cattails would develop. However, in a multi-cell operation and where the additional storage space is required lagoons have been successfully emptied as long as the cell can be refilled in a relatively short period.

Bottom

The pond bottom should be graded level for uniformity of water level control. Bentonite, asphaltic coating or other suitable material may be used to control the rate of percolation.

Dykes (Berms)

Compacted embankments of impervious material should be constructed with a minimum top width of eight feet and side slopes of four horizontal to one vertical for both inner and outer walls. The berms should have a freeboard of three feet above maximum water level and be erosion-protected with suitable grass covers. Additional protection for embankments such as rip-rap may be necessary.

Inlet Structure

Inlets to stabilization ponds should be located near the bottom centre of the pond to encourage rapid dispersal of incoming wastes. Immediate dilution of the wastes avoids the possible formation of local septic conditions or sludge bank deposits.

The influent may be discharged to the ponds either by forcemains or gravity sewers. Discharge inlets may be oriented in upward or horizontal directions for good mixing effects. Horizontal inlets are preferred for gravity flows. Where upward discharge inlets are used, the inlet should rise about one foot from the bottom and should not extend to such elevation that ice will damage the terminal structure during winter operation. The end of the discharge line should rest on a suitable concrete apron with a minimum size of two feet square.

Centre discharge inlets are employed almost exclusively in ponds of five acres or less. Ponds with larger surface areas are provided with inlets located at centre or third point. Inlet pipes into multi-cell ponds should be centre discharging but this does not apply to those cells following the primary cell in series operation.

Overflow Structure

The location of the outlet near the windward shore should prevent any wind induced short circuiting and premit maximum time-distance between inlet and outlet.

An effluent overflow structure is generally provided with an adjustable weir to permit variations in liquid depth between two to five feet, together with facilities for complete draining of the pond. An example of such controls is a stop plank support in a manhole for pond level control and an adjustable level intake structure to the manhole.

Multiple Ponds

The use of multiple cells to provide greater flexibility of operation is desirable. Multiple cells permit both series and parallel operation. Erosion is reduced. Also multiple cells permit rapid filling of the ponds and thereby prevent the growth of aquatic plants.

In general the tendency is to operate the ponds in series during the summer months and in parallel during the winter season. Parallel operation is required if the organic load is high since series operation would overload the first cell.

Fencing and Signs

The installation of fencing and "No Trespassing" signs is mandatory. The purpose of the signs is to notify persons of the nature of the facility and discourage trespassing. The minimum requirement for fencing is that it be stock-tight and at least six feet high to keep out animals and unauthorized persons.

Pretreatment

Usually, treatment of the raw sewage prior to application to the stabilization pond is omitted. Although some savings in land cost may be realized by reducing the BOD loading to the pond by preliminary treatment, the initial cost of a primary sedimentation unit plus the operating and maintenance costs usually offset any saving.

Normally, chlorination of the pond effluent is not required since in many cases seasonal discharge can be arranged for protection of the receiving waters during the recreational period. However, chlorination can be provided if necessary.

Aerated Lagoons

Although the main concern of this lecture relates to waste stabilization ponds some reference to aerated lagoons is necessary.

In many cases an adequate area for a conventional pond is not available due to the high organic load resulting from industrial wastes. Consideration is given to the use of supplying oxygen to the lagoons by mechanical means to reduce the area requirements. The continuous oxygen supply of the mechanical aerator, 24 hours a day, permits the aerated lagoon to handle more wastes per day per unit volume.

Aerated lagoons may be divided into three basic types depending on the removal processes associated with them. These are aerated - algae; aerobic - facultive; and complete mix aerobic.

Aerated Algae

Aerated algae lagoons with their shorter detention times (20 to 40 days) and higher loadings provide required oxygen 24 hours a day to permit deeper depths and to prevent odours. Mixing by the aeration device accelerates bacterial and algal activity and with higher loadings tends to grow more algae than conventional ponds per pound of BOD removed. Unless these algal cells are destroyed by higher life forms or settle to the bottom they are lost in the effluent.

In algal growth systems, therefore, putrescible organics are converted into stable inorganics by bacteria but they are in turn converted into putrescible algal cells which may enter the receiving stream to eventually exert an oxygen demand.

Aerobic Facultative

If the detention time is further decreased to four to fifteen days and the mixing intensity is further increased the number of algae cells may be minimized.

In aerobic-facultative lagoons aerobic conditions are maintained throughout the liquid but the aeration device does not provide sufficient solids suspension and solids settle out to decompose anaerobically on the bottom.

Complete Mix Aerobic

In the complete mix aerobic aerated lagoon the aerator must transfer oxygen, provide mixing to disperse the oxygen evenly in the whole basin and suspend the solids to prevent settling.

Detention times in the order of two to six days are used to obtain BOD removals of 60 per cent. The effluent suspended solids will be equal to the solids in the basin. This process is usually followed by a conventional pond.

Mechanical aerators, diffused air tubing and aeration guns have been employed as the aeration devices.

Like all waste treatment systems, the aerated lagoon has certain advantages and disadvantages. It appears to have some value where oxidation ponds are overloaded or where property is expensive, or when strong wastes are to be treated.

PUBLIC HEALTH CONSIDERATIONS

The same precautions which are used in the operation of conventional sewage treatment plants should be practised with waste stabilization ponds. Even though the reduction in bacteria in lagoons is

quite high, the possibility of an infection by contact with the sewage should be recognized. Thus, the need for adequate fencing to prevent access by children and animals as well as the posting of signs prohibiting trespassing, is apparent.

Ground water supplies, particularly those used for municipal purposes, should not be accessible to the seepage from stabilization ponds. Questions have also been raised regarding the possibility of infection being transported by wild fowl which frequent these ponds as well as the infection of livestock by watering in streams receiving lagoon effluent. As yet, no significant data have been established to substantiate these possibilities.

Surveys of mosquito breeding in lagoons have concluded that production will be of little consequence if weed growths are prevented or eliminated and larvicide is used as required, particularly, if difficulty occurs during initial filling.

ECONOMICS OF WASTE STABILIZATION PONDS

The feasibility of waste stabilization ponds depends largely upon the availability of suitable land. In many cases, land costs could be double or triple the completed lagoon construction costs before equaling the conventional plant cost.

Experience has also shown that in addition to providing advantages of a high degree of treatment, low initial capital cost and low operation and maintenance cost, the waste stabilization pond is quite flexible in areas which are subject to rapid population growth. It has been found that lagoons may be re-sited and constructed downstream and the pond area which has appreciated in value may then be reclaimed for housing or industrial site development. Also if a municipality outgrows the use of the lagoons, more than adequate area is available for using a mechanical plant.

In addition to being an economical method of sewage treatment, the waste stabilization pond has been useful in providing a polishing of conventional, primary or secondary treatment plant effluents. There are two municipalities in Ontario in which it has been found to be more economical to abandon existing conventional treatment works and construct waste stabilization ponds to handle the entire sewage flow from the community. In both instances, it was not possible to obtain an adequate area for the lagoons at the plant site but suitable land was found downstream at a reasonable cost.

Lagoon maintenance mainly consists of cutting of the grass on the dykes. The removal of scum, grease and floating material is generally no problem. Seasonal operation of valves may be required for multiple-cell installations or where variable depth control is exercised. Removal of emergent vegetation may be required periodically but this may be minimized by providing the required liquid depth.

Costs for lagoons are not always less. Many details such as longer outfall sewers, high pumping costs, high original land costs or difficult construction features may raise costs well above those for the conventional plant. However, a feasibility cost study will indicate the economical advantages or disadvantages of waste stabilization ponds for each installation.

CONCLUSIONS

In reviewing the accomplishments of waste stabilization ponds throughout the country, the following conclusions are apparent:

1. Research and field investigations have definitely proved that waste stabilization ponds are a practical and economical method of sewage treatment.

2. Properly designed and operated stabilization ponds may be expected to provide a degree of treatment comparable to conventional secondary treatment plants during part of the year.

3. In Ontario, in order to reduce the possibility of odours design loadings of 20 pounds of BOD per acre per day are being used.

4. The factors to be considered in site selection for waste stabilization ponds are essentially the same as conventional treatment plants except that a greater isolation distance is normally maintained.

5. There is no evidence to indicate that stabilization ponds constitute a public health hazard.